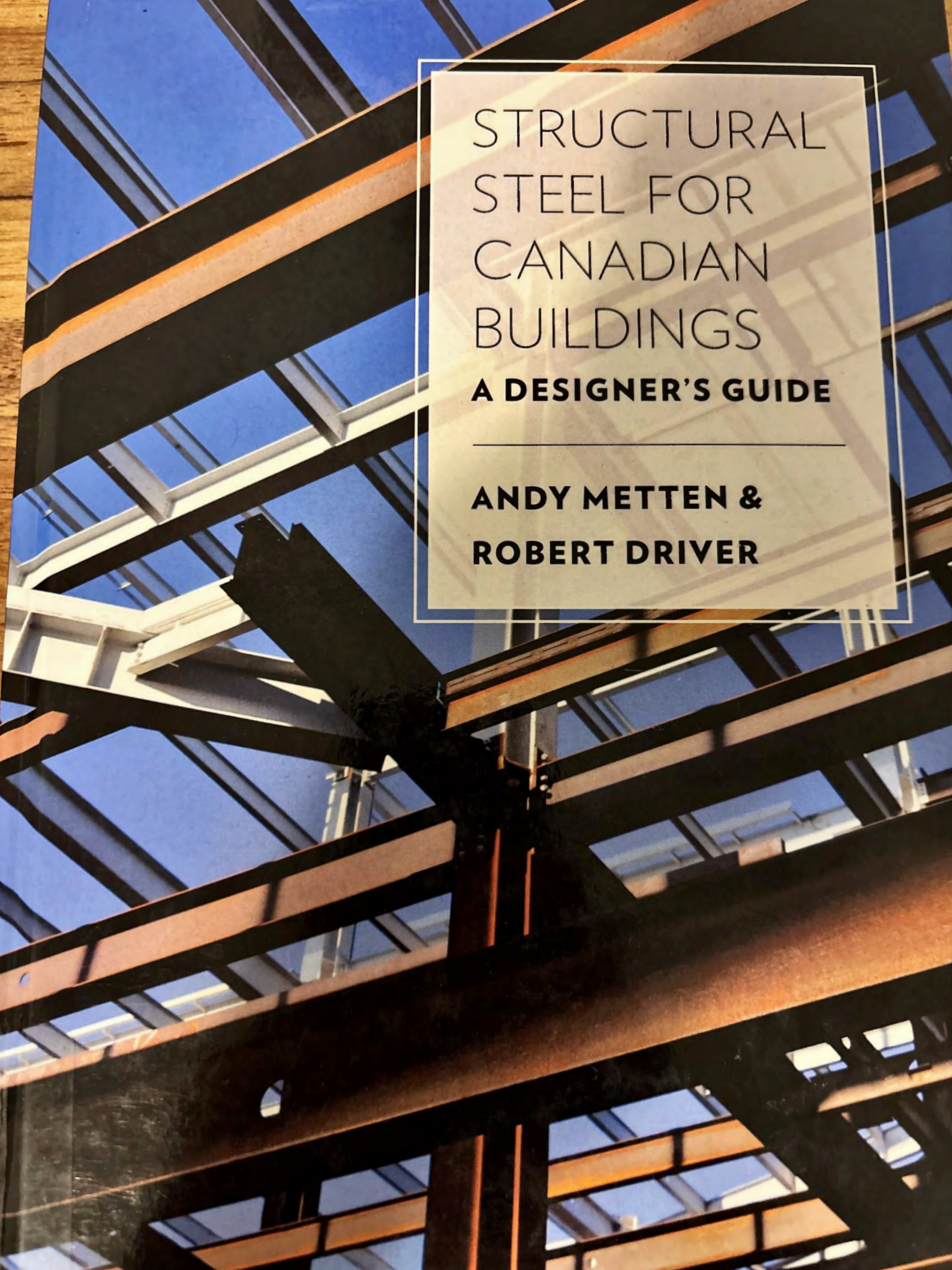


STRUCTURAL STEEL FOR CANADIAN BUILDINGS

A DESIGNER'S GUIDE

ANDY METTEN &
ROBERT DRIVER

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Andrew W. F. Metten and Robert G. Driver

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CHAPTER 1
INTRODUCTION TO STRUCTURAL STEEL DESIGN

Structural steel is a wonderful and versatile material for design, whether creating elegant and beautiful buildings or utilitarian sheds. Melting of iron for the production of tools and other small objects dates back to the Iron Age, more than 3,000 years ago. The iron-making process developed through ancient times, with blast furnaces for production of steel developing in many different parts of the world at times ranging from the first to the fifth centuries, though more modern and effective furnaces date from the 1500s. By the opening of the Iron Bridge across the Severn River in England in 1781, the production of structural steel was increasingly affordable and steel was increasingly used in structures throughout the Industrial Revolution. In Western Canada, the early use of structural steel in buildings is fundamentally tied to the building of the transcontinental railway, when larger railway bridge, station, and hotel structures were constructed of steel. Following the railway's completion, it provided a means of transportation for getting the steel from the furnaces in Eastern Canada and United States to the West Coast. By the early 1900s, use of structural steel in buildings across Canada was common.

Structural steel is an alloy of iron with other metal additives such as carbon. A fundamental property of the alloy, the carbon content in structural steel determines the material's strength but also its hardness and ductility, where steels with high carbon content are more brittle and more difficult to weld than those with a lower percentage of carbon. Addition of elements to the steel such as chromium, nickel, and tungsten can improve the corrosion resistance, toughness, or strength of the steel. The chemical composition of the steel is of interest to metallurgists, but from a structural standpoint, we are mostly interested in the certified grade of the steel and its mechanical properties.

1.1 ADVANTAGES OF STEEL AS A STRUCTURAL MATERIAL

There are many advantages to the use of structural steel on a project. Structural steel designs with exposed structural elements can be elegant and often are dramatic, with long-span roofs free of columns because steel has a high strength-to-weight ratio and a huge capacity in both tension and compression. Steel structures are lighter than comparable concrete structures, reducing their footing sizes and adapting them well to areas with poor soils. Lighter structural weight also reduces seismic loads. Further, lighter structural weight means that less of the area of the lower floors will be taken up by columns, making steel highly suitable for very tall buildings.

The ductility of steel is a huge advantage for the designer. As a ductile material, steel generally performs well under overload conditions and dynamic loading, including seismic and blast impacts, due its ability to redistribute load if well detailed. Figure 1.1 shows an actual stress-strain curve from a standard tension test of a coupon taken from a 9.52mm (3/8-inch) thick steel plate of G40.21 Grade 350WT steel. The ductility of this steel can be considered typical for structural grade steels, but rupture strains sometimes exceed 40%. The comparative proportions of the elastic and inelastic components of the tensile response are also evident in Figure 1.1. As a simple rule-of-thumb, the strain at the ultimate stress can be

estimated as nearly 100 times that at the yield point. The three short vertical lines on the yield plateau are where static yield stress values were taken by reducing the strain rate temporarily to zero, eliminating the effect of the rate of loading on the apparent material property. As is typical, the yield stress is significantly higher than the nominal value for this grade of 350MPa.

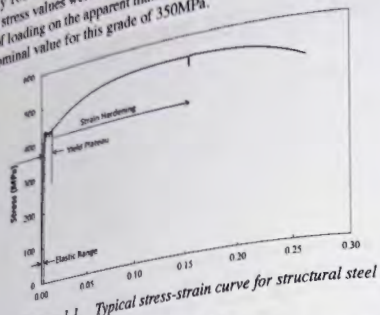


Figure 1.1. Typical stress-strain curve for structural steel

Other significant advantages of structural steel are as follows:

- Structural steel is generally fabricated off site, and fabrication can be performed while other site activities such as site preparation and footing construction are being performed. This can reduce the project construction time. Steel erection is often more rapid than erection of other framing systems.
- Most steel buildings do not require formwork or falsework, allowing erection of only permanent items and eliminating the need to construct and then remove temporary structures. This advantage proves valuable when constructing a new mezzanine inside an existing building. The new floor is built one piece at a time, using equipment that is of a weight suitable for the existing floor under the new mezzanine.
- Steel is sometimes considered to be a "green" construction material as it may be constructed from recycled materials and can be recycled following building demolition. Additional information on the green aspects of steel will be covered later in this chapter.
- In interior use, structural steel is a stable construction material and does not have the corrosion problems associated with the early post-tensioning systems used between 1940 and 1970. Steel is also not subject to attack from insects or shrinkage due to moisture change in an interior environment. Provided it is well protected against corrosion, steel will perform well in moist and wet conditions.
- From the standpoint of coordinating with the mechanical engineer it is easier to pass small ducts and conduits through the webs of steel beams than through concrete beams. This is especially true if it is desired to place a conduit through an existing beam.
- During construction, the placement of concrete on floors can be delayed until the steel deck above the concrete placement is completed. This can be of great assistance in getting a good concrete finish during wet winter weather.
- Steel is a non-combustible construction material. Larger buildings are usually required to have occupied floors constructed from non-combustible materials.
- For roofs of mechanical rooms, it is easy to attach pipe racks and other mechanical equipment using clamps attached to the flanges of the steel beam. This eliminates the need to drill.
- Much of the steel structure will be framed with simple span members, making it easier to adapt to differential settlement that may occur in soft soils.
- Steel is often used for structures in remote areas where delivery of concrete would require setting up a batch plant and it would be difficult to transport and house the necessary trade workers for a large concrete structure.

1.2 CHALLENGES OF STEEL AS A STRUCTURAL MATERIAL

Despite these many advantages, it is important for the designer to know the disadvantages of the material so that any drawbacks can be minimized in the design, as follows:

- One of the most significant disadvantages of steel construction when compared to concrete construction is that the depth of steel framed floors will often be greater than that required for reinforced concrete. For example, many apartment and small office structures can be constructed from concrete slabs of 6 to 9 inches thick, depending on the span. For similar spans in a steel-framed building, the floor will be 2½-inch concrete topping on 1½-inch metal decking and the steel beams below will be 10 to 12 inches deep, leading to an overall depth of 14 to 16 inches. The increased structural depth increases the cladding requirements on the building and length of stair runs. The greater floor-to-floor heights reduce the number of floors if the height of the building is fixed, and the longer stair runs will reduce the available space to sell or lease and may necessitate a mid-storey landing.
- The second disadvantage that must be resolved in most multi-storey steel-framed buildings is that steel for most occupied floors is required to be fireproofed. For most cases this includes columns, beams, and the underside of concrete slabs supported on metal deck. In some cases if the concrete topping on the metal deck is sufficiently thick it may be possible not to fireproof the underside of the deck but beams and columns will still require fireproofing.
- The designer of a steel building must pay more attention to the design of floors for vibration. Steel-framed floors designed only on the basis of strength can be bouncy and can lead to complaints from users. A variety of analysis methods have been developed to determine if a floor will be unpleasantly bouncy, but these methods are much more complex than the concrete system of satisfying minimum span-to-depth ratios.
- Another disadvantage is that the price of steel has more variation than concrete. The most common structural shape used in building design is the wide-flange section, but there are no producers of wide-flange sections in Canada. At present, all wide-flange shapes come from the United States or offshore and therefore are subject to variations in the Canadian dollar. At various times shortages of steel due to overseas demand for steel and scrap steel have resulted in significant price increases in the cost of steel building material during the past year. However, the cost increases for steel are also reflected in the cost of steel reinforcing for concrete structures and steel nails for wood ones. The cost of labour in the concrete and lumber framing sectors of the construction market has risen much sharper than has the cost of labour for steel construction as there are not the same acute labour shortages in the steel industry at present.
- The Vancouver construction market is well versed in building floors from reinforced concrete. This means that the cycle times for concrete floors are often less than a week and this takes away from the traditional time advantage enjoyed by steel. As a result, structural steel floors are used extensively in office buildings in the United States and Eastern Canada but only occasionally in Vancouver.
- When considered for residential or low-rise institutional construction, the underside of structural steel floors will not have a flat soffit and consequently a drop ceiling will be required in most architectural applications. Many residential projects have conduit or ducts in the slab which are difficult to accommodate in steel buildings as the slab thickness in steel construction is usually in the range of 65mm to 115mm, which does not allow sufficient room for placing conduit or ducts in the slab.
- Exposed structural steel in buildings usually requires painting, and painting of steel in exterior conditions will result in a need to repaint at intervals. The length of time between paintings will depend on the environmental conditions, including exposure to abrasion, exposure to sun and moisture and the quality of paint.

1.3 WORKFLOW VARIATIONS STEEL VS. CONCRETE

As a material distinct from concrete, several aspects of steel need to be considered when doing a design. The designer must keep in mind that steel sections are thinner than concrete sections and therefore more susceptible to buckling. The designer must constantly work to provide stability in the structure to mitigate this problem. Many of the design equations for member capacity will be controlled by buckling over at least part of their range. Steel roof diaphragms have less inherent strength and stiffness than concrete diaphragms, and more attention must be paid to making steel roof diaphragms work.

One significant advantage for the designer of a steel building is that steel beams usually need less detailing and drawing time than do comparable concrete beams. While both steel and concrete beams are shown in plan on design documents, it is common to elevate concrete beams. Steel beams are usually not elevated on the structural drawings and much of the actual detailing for the steel structure is carried out by the steel fabricator rather than the designer. Much of this detailing is now performed by sophisticated three- and four-dimensional computer programs, which build a virtual model of the structure, identifying conflicts and challenges prior to fabrication or erection. As we will see, the modern steel industry makes very sophisticated use of computers.

In terms of structural services, the design and field services of a steel building are usually quite different from those of a concrete building. The most fundamental difference is that field services in concrete buildings come mainly from site observations of the placed reinforcing steel, while for a steel building, much of the field services are performed by review of shop drawings prior to steel fabrication. Structural design of steel buildings can result in literally thousands of shop drawings that must be reviewed under a tight schedule. However, for structural steel buildings, this results in fewer field inspections by the designer than does reinforced concrete construction, where site visits must fall in that brief window between the time the reinforcing steel is complete and the time the concrete arrives on the site. In contrast, site visits in steel construction have a bit more timing flexibility.

1.4 GREEN ASPECTS OF STEEL AS A STRUCTURAL MATERIAL

Consideration of the environmental aspects of steel as a material is important in structural design as each material must compete on the basis of its environmental benefits, and materials will often be chosen on Leadership in Energy and Environmental Design (LEED)© Green Building™ projects on the basis of having a small environmental footprint. Steel is the material of choice and thus is used extensively in projects that reduce emissions of CO_2 such as wind farms and turbines for hydroelectric projects. Steel has multiple advantages as a green material, beginning with its manufacture and continuing on to structural steel fabrication and then to end-of-life and reuse or recycling.

In the manufacture of steel, there are two types of modern steel furnaces: the basic oxygen furnace, which uses 25% to 35% scrap steel combined with iron ore, and the electric arc furnace, which uses 90% to 100% recycled steel. Most steel produced in North America is manufactured using electric arc furnaces. Much of the steel produced in North America is produced using electric arc furnaces that use existing (post-consumer) steel as their raw material: old car bodies, dismantled buildings, abandoned appliances, recycled food containers, and other items that would otherwise be sent to landfills. Additionally, the manufacture of steel from scrap steel is energy efficient. Recycling is cost effective and working with clean recycled steel in electric arc furnaces, instead of virgin iron ore in blast furnaces and then basic oxygen furnaces, requires less energy (because it avoids mining, processing, transporting, and transforming, which produce large amounts of greenhouse gasses), and cuts water usage by 40% because iron ore isn't being cleaned and cooled.

Before construction, much of the fabrication of steel occurs in plants in areas of affordable housing, allowing fabrication work to be done by workers who commute small distances. Projects using other materials are more site-labour intensive and require the majority of the workers to drive to the site. Waste material generated in structural steel fabrication, including cut-off ends of beams, can be recycled by re-smelting. During construction, the material used in steel structures is generally part of the permanent structure. There is no need for wood forms as the composite metal deck is the formwork.

Because steel is a permanent and inert material, especially when protected in interior use, buildings constructed of steel can have a long life without exhibiting deterioration or sagging, and therefore will not require replacement as often as buildings built with other materials that exhibit long-term deformation under load. Once a project has come to the end of its life, the steel in that structure can be recycled by removal of individual elements and reuse in other structures as shown in Figure 1.2. Parts of steel buildings can be reused in other buildings; for example, during demolition carefully removed stairs or beams can be relocated to other buildings. On projects where individual steel elements cannot be reused, the steel can be separated from other elements of the demolition and smelted to make new steel products. The steel extracted from buildings and other structures is generally of good quality and can be used to create new structural steel. Steel is magnetic and can easily be separated from other waste by use of magnetic separators. Steel made from lower-grade scrap can be used in items that require lower strength, including steel studs and sheet steel products.



Figure 1.2. Steel recycling from building demolition is common, as shown in this 2007 photo taken in Vancouver. The steel beams, joists, columns, steel deck, and steel studs were all separated and recycled into new materials. With the exception of selective demolition at the start of the building removal, other construction materials are less likely to be recycled.

Designers should be careful not to fall into the trap of trying to specify that a certain percentage of recycled steel be used in the construction of a structure. The percentage in recycled content is not given on mill certificates and it is not possible to enforce that a given quantity of recycled steel be present. At best, steel mills can help with writing letters of assurance that state approximate quantities of recycled steel used in the mill on a historic basis. Most large projects will have steel coming from several mills, and a rigorous pursuit of such letters may prove to be a challenge.

1.5 AVAILABILITY OF STEEL STRUCTURAL MATERIAL

It is important when doing a design to consider the availability of the steel material and the section shapes to avoid delivery delays and additional costs because a rare section size will be harder to get. A variety of standard steel sections are commonly available, and these different shapes work well for different purposes. Typically available section shapes and their uses are shown in Table 1.1.

Table 1.1. Typically available steel section shapes

SECTION TYPE	USE IN BUILDING STRUCTURES
Wide-flange Sections	Basic beam shape Column sections
Welded Wide Flange	Girders
Channels	Stair stringers Short, lightly loaded beams Trimmer beams at mechanical openings Wind girts
HSS (Hollow Structural Sections) (Round, square, or rectangular)	Trusses Columns Bracing Handrails
Angles (Equal or unequal legs)	Diaphragm chords Trusses Built-up sections Lintels / Façade support
Tees (Cut from wide-flange shapes)	Trusses Canopies and decorative steel Built-up members
Plate	Base plates Connecting elements Built-up members
Bar	Connector elements Built-up sections
Pipe	Handrails
Rod	Hangers Wind bracing in non-seismic zones

Sometimes the shapes that are used in design are a combination of the sections in Table 1.1; however, it is more economical if a single section can do the work of a combined section.

1.6 FLAT-BAR STEEL AS A BASIC BUILDING BLOCK

Flat-bar steel is useful in connector elements but only comes in certain sizes. Commonly available sizes are shown in Table 1.2. If the weight is not listed, then that flat-bar section is hard to get.

Table 1.2. Typically available flat-bar steel section sizes

WEIGHT OF FLAT BAR IN POUNDS PER LINEAR FOOT – COMMON SIZES												
	FLAT BAR THICKNESS IN INCHES											
Width	**¼ in**	**⅜ in**	**½ in**	**⅝ in**	**¾ in**	**⅞ in**	**1 in**	**1¼ in**	**1½ in**	**1¾ in**	**2 in**
½ in	0.213	0.319	0.425								
⅝ in	0.266										
¾ in	0.319	0.478	0.638		0.956	1.28					
1 in	0.425	0.638	0.850	1.06	1.28	1.70	2.13	2.55				
1¼ in	0.531	0.797	1.06	1.33	1.59	2.13						
1½ in	0.638	0.956	1.28	1.59	1.91	2.55	3.19	3.83	5.10			
1¾ in			1.49		2.23	2.98						
2 in	0.850	1.28	1.70	2.13	2.55	3.40	4.25	5.10	6.80			
2¼ in			1.91									
2½ in	1.06	1.59	2.13	2.66	3.19	4.25	5.31	6.38	8.50			
3 in	1.28	1.91	2.55	3.19	3.83	5.10	6.38	7.65	10.20	12.75	15.30	
3½ in	1.49	2.23	2.98		4.46	5.95	7.44	8.93	11.90			
4 in	1.70	2.55	3.40	4.25	5.10	6.80	8.50	10.20	13.60	17.00	20.40	27.20
4½ in		2.87	3.83		5.74	7.65						
5 in	2.13	3.19	4.25		6.38	8.50	10.63	12.75	17.00			
5½ in												
6 in	2.55	3.83	5.10	6.38	7.65	10.20	12.75	15.30	20.40	25.50	30.60	
7 in			5.95		8.93	11.90		17.85				
8 in			6.80		10.20	13.60	17.00	20.40	27.20			
10 in			8.50		12.75	17.00		25.50				
12 in					15.30	20.40		30.60	40.80			

Other sizes can be cut from plate, but designers should use the sizes above where possible. The sizes with no weight given are not commonly available and alternate sizes should be selected.
Material is CSA G40.21-44W or 50W (300MPa or 345MPa).

If the least dimension of a desired piece of flat steel is to be greater than 12 inches, then a plate will be required. Taking a shape from a plate will require at least two cuts while a flat bar needs only one cut to get the size wanted.

1.7 PLATE MATERIAL AVAILABILITY

Plates are used where flat-bar sizes are too small or where the required width is between standard flat-bar sizes. Plate material comes in 350W and 300W steel and is easiest to obtain in imperial thicknesses. The use of metric thicknesses recommended in Part 6 of the *CISC Handbook* may result in substitutions or long delivery times. Plate thicknesses from Table 1.3 should generally be used. It is advisable to obtain advice from local fabricators.

Table 1.3. Typically available steel plate sizes

Inches	mm	Inches	mm	Inches	mm	Inches	mm
0.1875	4.8	1.125	29	2.25	57	6.00	152
0.2500	6.4	1.250	32	2.50	64	7.00	178
0.3125	7.9	1.500	38	2.75	70	8.00	203
0.3750	9.5	1.750	44	3.00	76	9.00	229
0.4375	11.1	2.000	51	3.50	89	10.00	254
0.5000	12.7			3.75	95	11.00	279
0.6250	15.9			4.00	102	12.00	305
0.7500	19.1			4.50	114		
1.0000	25.4			5.00	127		

Source: Rob Third of George Third and Sons

1.8 STEEL BEAM AVAILABILITY

The *CISC Handbook* lists many steel sizes, some of which are not readily available. In the past, the sections that were commonly available were highlighted in yellow in the steel manual, but this practice was dropped in the 7th edition. The 10th edition of the *CISC Handbook* uses a very light grey for nine sections that were once commonly available from Algoma Steel (e.g., W610x84) but are now not readily produced. Some steel designers will approach the steel fabricators at the start of a project and request a list of sections that can be obtained without long pre-order delays. If the designer chooses steel sections from this list, then the design will result in slightly higher steel weights but will cause fewer delivery delays. A possible list is shown in Table 1.4, with the caution that this list will change depending on the location in the country, time, size of order, and a multitude of other factors. The 11th edition of the *CISC Handbook* is expected to provide improved guidance on commonly available shapes and plates that recognizes the challenges with respect to geographic and time-dependent variables.

Table 1.4. Typically available steel beam sizes

	Usually Available Wide-flange shapes											
		<——Weight Range (kg/m)——>										
	*common additions 102mm flanges	20-30	30-40	40-50	50-60	60-70	70-80	80-90	90-100	100-110	110-120	120 Plus
W150	W150x14 W150x18	W150x22	W150x30 W150x37									
W200	W200x15 W200x19 W200x25 W200x22	W200x27	W200x31 W200x36	W200x42 W200x46	W200x52 W200x59		W200x71	W200x86				
W250	W250x18 W250x22 W250x28		W250x33 W250x39	W250x45 W250x49	W250x58	W250x67	W250x73	W250x80				
W310	W310x21 W310x24 W310x28 W310x33		W310x39	W310x45	W310x52	W310x60 W310x67	W310x74 W310x79	W310x86	W310x97	W310x107		
W360			W360x33 W360x39	W360x45	W360x51 W360x57	W360x64	W360x72 W360x79					
W410			W410x39	W410x46	W410x54	W410x60 W410x67	W410x74					
W460					W460x52	W460x60	W460x74	W460x82 W460x89				
W530						W530x66	W530x74		W530x92	W530x101 W530x109		W530x123
W610								W610x82	W610x92	W610x101	W610x113	W610x125 W610x140 W610x155 W610x174 W610x195

1.9 MATERIAL GRADE AVAILABILITY

Material grades that are commonly available are given in Table 1.5.

Table 1.5. Typically available material grades

Item	230MPa	240MPa	300MPa	350MPa	Made in Canada	Remarks
Wide Flanges				X	No	
WT sections				X	No	
Welded Wide Flanges			X	X	Yes	(Note 5)
Angles			X	X	Yes	(Note 2 & 10)
Channels			X	X	Yes	(Note 10)
Plate			X	X	Yes	(Note 4 & 10)
Flat Bar			X	X	Yes	(Note 10)
Hollow Structural Sections				X	Yes	(Note 1)
Steel Deck	X				Yes	
Steel Studs			X	X	Yes	
Pipe		X			Yes	(Note 3)
Steel Rod			X		Yes	

Notes

1) Use HSS G40.21 350W Class C. Never use HSS Class H. Class H is difficult to source and it is difficult to verify if Class H sections have been used. When using round HSS use ASTM A500 sections (and the appropriate tables in the *CISC Handbook*) as getting G40.21 Class C sections in round is difficult. The yield stress of ASTM A500 round sections is 317MPa. The new grade ASTM A1085 is considered equivalent to 350W and superior to A500.
2) Some steel books contain metric-sized angles, but these are no longer produced in North America and should not be specified.
3) Pipe may come with yield strengths greater than 240MPa, but with the exception of seismic design, it would be reasonable and conservative to assume a strength of 240MPa.
4) Plates are available in metric or imperial thicknesses, but preference should be given to using imperial thicknesses due to their higher availability (10mm plate will often be 3/8 inch or 9.5mm). Plates are also available in 350W, but it is typical to design using 300W, as this size is more frequently stocked and it is usually of no consequence if the higher strength plate is substituted.
5) As welded wide-flange sections are fabricated from plate, they can be specified in either 300W or 350W. If the project uses welded wide flanges, then the general notes on the structural drawings should specifically give the required yield strength.
6) It is not permitted to go to a higher yield stress than the specified minimum value of the material that steel is certified for on the mill certificate when evaluating beams. Cutting sections from a beam and having these sections tested as a means of increasing the rated capacity of the beam is not permitted. It is also not permitted to use the yield value on the mill certificate when checking the capacity of the beam. If the mill certificate states that the material is 350W G40.21 then the designer must use Fy = 350Mpa, not the yield stress that is listed on the mill certificate.
7) The specified minimum tensile strength, F_u, of both 350W and 300W steel is 450MPa. This value is used in connection design and evaluation of sections near holes. The value of F_u for several types of steel can be found in Table 3-5 of the *CISC Handbook*.
8) Steel used in most building projects is specified to be W for weldable grade and to satisfy CAN/CSA G40.21 (Structural Quality Steels). In addition to weldable grade steel, there are other types including:

a) Type WT **W**eldable notch **T**ough steel for low temperature
b) Type R Atmospheric corrosion **R**esistant steel
c) Type A **A**tmospheric corrosion resistant weldable steel
d) Type AT **A**tmospheric corrosion resistant weldable notch **T**ough
e) Type Q **Q**uenched and tempered low alloy steel plate
f) Type QT **Q**uenched and tempered low alloy notch **T**ough plate

Further information on these steels, including their chemical contents, can be found in Part 6 of the *CISC Handbook*. Care should be taken in specifying these steels as only a limited number of structural shapes and sizes are produced in steel other than weldable grade. Delivery times for steel other than weldable grade will be longer and may require special mill orders with a minimum 150 tonne requisition. On small jobs the designer should seriously consider using means other than rare steel types to effectively execute the design.

1) Stainless steel is sometimes used in buildings as trim and other non-structural elements, though some uses such as handrails may require the designer to compute resistance capacity. Stainless steel has a minimum of 10.5% chromium which increases its resistance to corrosion. There are a variety of stainless steel alloys, the most common of which is 304. Strength varies by alloy selected; 304 has a yield of 200MPa. Stainless steel sections are limited in size and for hollow sections generally have thin walls, and the designer should check that the size being called up is available; specifying the equivalent HSS size in stainless steel will not be successful.
2) For several years after the yield strength on wide-flange members went to 350MPa the yield strength on plate, flat bar, channels and angles remained at 300MPa. To ease stock challenges and satisfy the most customers with a single rolling, the mills will now use a steel that is certified for several grades including both 300MPa and 350MPa. The material has a useable yield strength in design of 350MPa and so angles and channels should now take advantage of that strength and be called up as 350MPa.

1.10 FIRE RATING OF STEEL STRUCTURES

An important aspect of steel design is the need to provide fire rating to occupied floors. Elevated temperatures have a profound effect on the strength and stiffness of steel, including:

- Reduction in strength (50% reduction at 600°C)
- Reduction in stiffness (70% reduction at 600°C)
- Increase elongation (if a beam is heated by a fire from below then this may push apart columns and increase P-Δ effects)

These physical changes can reduce the capacity of the steel to the point where it fails to carry its self-weight and collapses. The most dramatic example of this is the failure of the World Trade Center Towers in New York in 2001 where failure occurred not from impact but from fire.

There are two types of protection for fires in buildings; these are active, which include the sprinkler system, and the passive, which include the use of non-combustible structural materials and fire-spray on the beams and columns to help them resist the fire load. Both the active and the passive fire protection can have an influence on the structural aspects of the building design. The active protection may require sprinkler holes in the webs of beams and the passive protection may require us to use members with higher M/D ratios and thicker topping on the metal deck than are required strictly by structural analysis.

To protect against failure due to fire, building structures are assigned a fire rating depending on the type of construction and the fire-proofing and other details. These fire ratings give the structure a time to collapse under standard fire conditions. An occupied floor in a large building will usually be required to have a two-hour fire rating, while the roof in that building (which is not considered to be occupied) will not be required to be fire rated. Care should be taken when designing a building with unusual framing that the unrated roof is not holding up areas of floor that are required to be rated.

When it comes to fire rating the most important value in the steel section tables is the M/D ratio. M/D ratio is the mass of the structural steel element (in kg/m) divided by the fire exposed perimeter (in meters) of the steel section. The fire rating may have minimum beam and column sizes and substitutions can be made to those sizes provided the same M/D ratios are obtained. M/D ratios can be found in Part 7 of the *CISC Handbook* in both metric and imperial units. There are different values for beams, which are assumed to have their top flange protected by the slab, and columns that are assumed to have fire on all sides. M/D ratios for HSS columns are not given in the *CISC Handbook* and must be computed by hand. M/D ratios for wide-flange beams are shown in Table 1.6.

Table 1.6. M/D ratios for wide-flange beams

DESIGNATION	SI M/D (KG/M / M)		IMPERIAL M/D (LB/FT/IN)	
	BEAM	COLUMN	BEAM	COLUMN
W200				
x42	47.6	40.0	0.813	0.683
x36	41.4	34.8	0707	0.594

The fire ratings are given in tests performed by recognized laboratories (ULC and NRC in Canada) that base their findings on tests of standardized fires under the "assembly" being followed. A rating of a system is given in terms of number of hours to collapse. For a steel structure assembly, the rating may contain the following requirements:

- Minimum M/D ratios for columns and beams.
- Minimum fire-spray requirements for columns and beams.
- The thickness of concrete cover over the flutes of the deck if the deck is to avoid the need of fire-spray. (This is often 115mm but may vary depending on the assembly requirements.)
- Require that the deck concrete contain reinforcement.
- If the assembly is considered "restrained" or "unrestrained."

The requirements of minimum concrete thickness on the deck and minimum M/D ratios will have an influence on the structural design of the building. The weight of fire-spray should be included in the building design loads but is often considered part of the 1kPa of "Partitions / architectural finishes."

Required M/D ratios should be discussed with the architect and the Code consultant at the start of the job. If the HSS columns that are buried in Gyproc-covered walls need to have a minimum M/D ratio that depends on the number of layers of Gyproc added around the columns, it is desirable to know this early in the design. The most common fire-rated Gyproc is 5/8 inch (15.9mm) type X Gyproc. Extracting the cover requirements from Appendix D to Division B of NBCC, we find the M/D ratio for steel columns as shown in Table 1.7.

Table 1.7. Fire resistance requirements for steel columns

Fire Proofing	Minimum Thickness of Type X Gyproc	1 Hour Fire Resistance Rating	2 Hour Fire Resistance Rating
1 layer 5/8 inch Type X	15.9mm	M/D = 55	Not Possible
2 layer 5/8 inch Type X	31.8mm	M/D = 35	M/D = 75

Source: Table D-2.6.1.5 of Appendix D to Division B for NBCC 2010

HSS 127x127 columns have M/D ratios shown in Table 1.8.

Table 1.8. HSS 127x127 M/D ratios

COLUMN	PERIMETER	MASS	
	(m)	kg/m	M/D
HSS 127x127x4.8	0.492	17.9	36.4
HSS 127x127x6.4	0.486	23.2	47.7
HSS 127x127x8.0	0.481	28.4	59.0
HSS 127x127x9.5	0.475	33.3	70.1
HSS 127x127x13	0.464	42.3	91.2

Note that all the sections are an HSS 127 x127. This is because the corners of the HSS are rounded, with the rounding being more pronounced for a thicker wall.

Combining these charts, we find the requirements for an HSS column located in a 6-inch wall with one or two layers of 5/8 inch Type X gypsum wall board to be as shown in Table 1.9.

Table 1.9. Minimum wall thickness of HSS 127x127 steel columns to satisfy fire resistance ratings

Fire Proofing	Minimum Thickness of Type X Gyproc	1 Hour Fire Resistance Rating	2 Hour Fire Resistance Rating
1 layer 5/8 inch Type X	15.9mm	HSS 127X127 x 8.0	Not Possible
2 layer 5/8 inch Type X	31.8mm	HSS 127x127x4.8	HSS 127x127x13

To highlight the importance of fire in steel structures S16 contains Commentary K "Structural Design for Fire Conditions." Included in this commentary is a very clear definition of "restrained" and "unrestrained," which is an important characteristic of the floor assembly when evaluating fire assemblies. This is useful as many Code consultants are unsure of the difference, while most of the fire assembly ratings for floors refer to restrained and unrestrained construction.

K3.2 Restrained Construction

For floor and roof assemblies and individual beams in buildings, a restrained condition exists when the surrounding or supporting structure is capable of resisting actions caused by thermal expansion throughout the range of anticipated elevated temperatures.

Steel beams, girders, and frames supporting concrete slabs that are welded or bolted to integral framing members (*i.e.*, columns, girders) shall be considered restrained construction.

K3.3 Unrestrained Construction

Steel beams, girders, and frames that do not support a concrete slab shall be considered to be unrestrained unless the members are bolted or welded to the surrounding construction that has been specifically designed and detailed to resist actions caused by thermal expansion.

A steel member bearing on a wall in a single span or at the end span of multiple spans shall be considered unrestrained unless the wall has been designed and detailed to resist the effects of thermal expansion.

If steel is to be exposed for architectural reasons and is needed to be fire-rated, then intumescing paint may be necessary. This paint expands to give a fire rating. It is, however, expensive, often more expensive than the steel itself, must be applied in several coats and must be tested to verify a suitable thickness is present. The paint leaves a heavy coating that is not as smooth as regular paint and may not be completely

pleasing to the eyes of the architect. Minimum M/D ratios will usually be required for steel that is fire rated with intumescing paint.

M/D ratios for HSS columns cannot be found directly in the *CISC Handbook* and to compute M/D for an HSS column, use the section properties table and take the mass in kg/m and divide by the surface area in m^2 /m as has been done in Table 1.10.

Table 1.10. M/D ratios for square HSS sections G40.21

		<---------- WALL SIZE FOR HSS (G40.21) SQUARE ---------->						
		16	13	9.5	8	6.4	4.8	3.2
HSS SIZE (SQUARE HSS G40.21)	305x305	119.8	95.8	72.7	61.2	48.9		
	254x254	118.6	95.7	72.5	60.8	48.9		
	203x203	116.5	94.5	71.9	60.4	48.5		
	178x178	115.1	93.7	71.5	60.1	48.5	36.7	
	152x152		92.6	70.9	59.8	48.1	36.6	
	127x127		91.2	70.1	59.0	47.7	36.4	
	114x114		90.1	69.6	58.8	47.6	36.3	24.4
	102x102		88.7	68.7	58.3	47.3	36.2	24.4
	89x89			67.8	57.6	46.7	36.0	24.2
	76x76			66.5	58.3	46.3	35.8	24.1
	64x64				55.5	45.7	35.1	24.0
	51x51					44.5	34.5	23.7
	38x38						33.4	23.3

What we see from this chart is that for Square HSS using G40.21 we need the following:

a) For a 2 hour rating with 2 layers of Type X Gyproc, minimum M/D = 75 provided only by a 13mm or 16mm wall.
b) For a 1 hour rating with 1 layer of Type X Gyproc, Minimum M/D = 55 provided only by a wall thickness of 8.0mm or greater.
c) For a 1 hour rating with 2 layers of Type X Gyproc, Minimum M/D = 35 almost all sections meet this with exception of those sections with walls of 3.2mm and 51x51x4.8 or 38x38x4.8 which are usually too small to consider as columns.

If a one-hour fire rating is required it will be less costly to use an HSS with the required wall thickness than to wrap each column in two layers of Gyproc. Wrapping in two layers of Gyproc may also make the wrapped column not fit in the wall as columns are often only 25mm less than the stud size used for the wall and two layers of Type X Gyproc uses up 32mm.

1.11 THE STRUCTURAL DESIGN PROCESS

The aim of the structural design process is to produce a set of drawings that clearly shows how to construct the structural system for the building in a manner that will safely resist the loading specified by the Code. The drawings must be clear as they will be read and interpreted by the building authority, the contractor and other engineers trying to modify the building in the future and, most importantly, by the client.

When designing a particular element or area of framing, the aim of the structural designer is to find a structural system that satisfies all the required structural criteria, fits the constraints imposed by the project, and is the most economical solution. The most economical structural solution is usually defined as being the one that uses the least weight of structural steel – this is a very crude measure that is used by

the quantity surveyor to compare the efficiency of designs. As all steel fabricators will tell you, minimum steel weight does not necessarily constitute the minimum cost.

When doing the design, we usually start by a determination of the governing structural forces in the member: for a beam this would be the factored bending moment, for a column it would be the factored axial load and for a beam-column it would include combinations of moment and axial forces. We would then pick the minimum weight element that has the capacity to resist this load. For a beam we will select the minimum weight beam that satisfies the criterion that its factored moment resistance exceeds the factored moment on the beam. We then must check other criteria, such as the deflection, before determining whether that the element is acceptable. Selecting an element at random, and then seeing if that element meets all the structural criteria, will seldom produce an economical design. The intent of this book is to illustrate the structural design process for structural steel buildings.

1.12 THE ART OF STRUCTURAL DESIGN

The process of taking a building from conceptual design to final inspection may take several months or even years. During that process it is not unusual for the entire concept of the building to evolve several times.

The process will usually involve interaction with consultants from disciplines including architectural, mechanical and electrical and cost consultants. While the aim of each of these consultants is to complete the building, their short-term aims may conflict with the designer's structural aims. For example, the designer may determine that deeper joists are more economical from a structural standpoint; however, the additional cost of cladding and building height to accommodate these joists may outweigh the structural savings. Many of the problems the designer will encounter in the structural design are not significant technical problems but are conflicts in the vision of the team members that must be solved with diplomacy and tact.

On any project, the need for well-organized design notes that can be followed at a later date (and possibly by a colleague not originally involved in the design) is essential. Organizing design notes into a three-ring binder divided into sections with topics such as "Structural Loads," "Mezzanine Floor Design," "Roof Design," and "Column and Footing Design" is a minimum. Design sheets should be dated and include a title at the top of each page that clearly explains what is being examined. For example, the title "Deck Design for Mezzanine Floor" shows at a glance what is being attempted and is much easier to understand than an untitled series of seemingly random calculations. Where Code requirements or handbook information is referenced, it should be noted (e.g., "Review of tension resistance follows S16 Clause 13.2"). This makes future comprehension of the notes easier and shows the reader that appropriate Code clauses were reviewed and followed. Notes should include sketches produced during the design process that show the structural configuration of the items being designed. A figure with a small section of floor and one of the beams labelled "Beam A" makes it much easier to determine at a later date what beam was being designed. This kind of labelling is vital when tracing back to see if a beam has sufficient capacity to pick up some extra load that it may be required to carry.

The actual design order of the parts designed will vary; for example, one designer may elect to design the lateral system before designing the columns. It is likely that the designer will have at least a concept design of the complete building, including a feel for approximate sizes, prior to fully completing the design and detailing of any of the building components. The complete design of a building may appear to be a daunting task, but broken into manageable pieces, the task can be tackled in an efficient manner.

The preparation of structural drawings and the coordination of the job with the architect and other disciplines is usually an ongoing task and is seldom left to the end of the project. Many design issues will only become apparent as the drawings are prepared.

The final review and an independent concept review are required by the *Code of Ethics of the Association of Professional Engineers and Geophysicists of British Columbia*. An independent concept review will result in a better design and more readable drawings containing fewer mistakes.

1.13 THE BUILDING CODE

The model building code in Canada is the National Building Code of Canada (NBCC) and all other building codes are derived from that code, with minor regional differences. The City of Vancouver has a unique charter that allows it to have its own building code, so it produces the Vancouver Building Bylaw. Apart from the City of Vancouver and areas under federal jurisdiction such as YVR and National Defence projects, the building code in effect in the rest of British Columbia is the British Columbia Building Code (BCBC), which is essentially the National Building Code of Canada adopted by the local jurisdiction. From a structural loading standpoint the differences among the three building codes used in British Columbia are subtle at best. For example, the Vancouver Building Bylaw requires a slightly higher loading for bedrooms and requires that $C_w = 1$ when doing snow load calculations. The design documents should clearly reference the correct building code for the *Authority Having Jurisdiction* or the local building authority in the location of the project. The drawings for a project submitted for building permit in the City of Vancouver will be rejected in the permit process if structural drawings reference the National Building Code of Canada or the British Columbia Building Code.

The National Building Code is usually updated every five years and the most recent edition is 2015. In addition to major revisions, the National Research Council issues revisions to the National Building Code of Canada that are picked up slowly by the Vancouver Building Bylaw and the British Columbia Building Code. It may take from one to four years after the NBCC is published for the provincial codes to come out.

From the structural designer's standpoint, two sections in the Code are of particular interest. The first is Part 4, which sets out the structural loads and procedures for determining the forces on the building. Here the designer will find how to compute the live loads on the floors, as well as the seismic and wind procedures. The second part of the Code that is important is Part II, which sets out the referenced standards.

1.14 STEEL DESIGN STANDARDS

The current (11th) edition of the *CISC Handbook* also contains the steel design standard, CAN/CSA S16-14. About every five years this standard will be updated. The previous edition was S16-09 and the next edition will be S16-19. When new standards come out, they often take longer to be accepted by the B.C. and Vancouver codes than by the national Code; this means that the old edition of the steel standard is in effect in some jurisdictions in British Columbia. This book will refer to the S16-14 throughout as it will be adopted by these other jurisdictions over the next 12 to 18 months. Recent editions of the steel standard are shown in Table 1.11.

Table 1.11. Steel design standards for various jurisdictions

LOCATION	CODE IN EFFECT (JANUARY 2016)	REFERENCED STEEL STANDARD	CODE ADOPTED
City of Vancouver	Vancouver Building Bylaw 9413 (2007) (based on NBCC 2010)	S16-09	December 2014
Federal Jurisdiction, including YVR	National Building Code of Canada 2015	S16-14	Early 2016
All other areas in British Columbia	2012 British Columbia Building Code (based on NBCC 2010)	S16-09	December 2012

In August 2014 the new steel standard S16-14 was released. The aim of this book is to not only make the designer familiar with S16-14 but to highlight its application to structural steel design.

1.15 LIMIT STATES DESIGN

The steel design standard, S16, follows *Limit States Design*. We will discuss limit states design throughout this book in relation to aspects of steel design, but it is important to know early on that several limit states must be satisfied. These consist of ultimate limit states that include exceeding the load carrying capacity and serviceability limit states that include vibration and deflection. Ultimate limit states are examined using load factors applied to the loads, while serviceability limit states use unfactored loads.

The majority of design time will be spent satisfying the ultimate limit state of not exceeding the load carrying capacity either through yielding or buckling and the serviceability limit state through deflection. The limit states design process involves determining the loads on the element and multiplying these loads by load factors given to us by the Building Code, thus producing the factored load on the member. The factored resistance of the member is determined by calculating its nominal resistance, then multiplying by a capacity reduction factor, Ø, to give the factored resistance. The factored resistance of the element must be greater than its factored load. The load factors represent the uncertainty and probabilistic nature of the loads, while the capacity reduction factors (resistance factors) represent the uncertainty and probabilistic nature of the resistance.

This is shown as follows:

Factored Resistance of member ≥ Factored Load on member

Ø*(Calculated Resistance) > Load Factors*(Calculated Loads)

E.g., $Ø*(F_y*Z) > (1.25*\text{Dead Load} + 1.5*\text{Live Load})$

1.16 LIMIT STATES FOR STEEL DESIGN

Clause 6 of S16 tells us that there are several types of limit states that the designer must guard against. These can be broken into ultimate limit states and serviceability limit states, as follows:

A) Ultimate Limit States
   1) Strength
   2) Overturning
   3) Sliding
   4) Fracture (the fatigue limit state is a state of crack propagation)

B) Serviceability Limit States
   1) Deflection
   2) Vibration
   3) Permanent deformation

1.17 ULTIMATE LIMIT STATES

In general the Building Code protects ultimate limit states by applying load factors to increase the expected loads and uses capacity reduction factors (Phi or Ø factors) to reduce the expected capacity. We then design to satisfy the most important equation in structural design:

Factored Resistance ≥ Factored Loads

This is expressed graphically in Figure 1.3, which shows the desired relationship between the factored load and the factored resistance. Both the factored load (for example, maximum snow load per year) and resistance (for example, moment capacity of the beam) are bell curves with a mean value. The desire is to have the area where the factored load exceeds the factored resistance be sufficiently low.

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This is expressed graphically in Figure 1.3, which shows the desired relationship between the factored load and the factored resistance. Both the factored load (for example, maximum snow load per year) and resistance (for example, moment capacity of the beam) are bell curves with a mean value. The desire is to have the area where the factored load exceeds the factored resistance be sufficiently low.

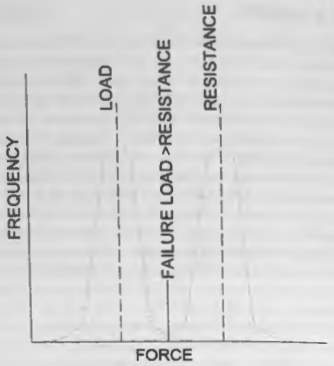


Figure 1.3. Desired relationship between the factored load and the factored resistance

If the expected capacity with Ø factors applied exceeds the factored loads, then the ultimate limit state is satisfied. However, we must be careful to determine that the capacity of the member or the structure is being evaluated correctly. For example, it is necessary to take account of the lateral restraint of cantilever beams if their failure method is to be correctly assessed.

In building design, other limit states such as overturning and sliding are generally foundation issues to be resolved by careful interaction with the soils consultant.

While fatigue limit states are closely analyzed in bridge construction, they are usually not studied in building design except where crane rails are involved. Fatigue issues are usually a function of the magnitude of the stress range and number of cycles of loading on the element during its life. Elements that have a large range of tensile stress between maximum and minimum stress, and those that undergo many cycles of stress fluctuations, are most at risk. In building design, the real live loads are often much smaller than both the design live loads required by the Building Code and the permanent dead loads. This means that the stress ranges on the members due to fluctuations in live load are generally low. For these reasons fatigue criteria are seldom observed in the design of buildings. However, the steel standard is used for the design of several projects, including crane components where fatigue must be considered, and the reader is directed to those sections in the steel standard and its appendices for additional guidance.

The development of load combinations and capacity reduction (Ø) factors is a complex science involving statistics on the mean and standard deviation of loads and resistances, as well as a judicious calibration to past experience. A lengthy review of the Building Code–mandated load factors was undertaken by the NBCC task force on wind and snow loads and modifications were incorporated into NBCC 2005.

Current Canadian building codes use a "Companion Action" approach to combining loads. Under this format there are "principal loads" that have load factors of 1 or greater and these are combined with "companion loads" that have a load factor of less than or equal to 1. The companion action approach has made combinations involving more than one variable load more logical. What the traditional load combinations did was say the wind and the snow made an agreement to only go to 70% of their full value if the other was present and there was no consideration of wind being present when the full snow load was present. Under companion action we look at the full snow load and then add in the amount of wind that is going to be present in that rare circumstance when full snow is present. A load factor less than 1.0 usually modifies the companion load. The companion action approach to load combinations is a change

in philosophy that reflects closer the real presence of loads on the structure. The load combinations are presented in Table 1.12.

Table 1.12. Load combinations required to be considered under NBCC 2015 for buildings without crane loads. For buildings with crane loads a more complex set of loads combinations exists

CASE	LOAD COMBINATIONS	
	PRINCIPAL LOADS	COMPANION LOADS
1	1.4D	-
2	(1.25D or 0.9D) + 1.5L	1.0S or 0.4W
3	(1.25D or 0.9D) + 1.5S	1.0L or 0.4W
4	(1.25D or 0.9D) + 1.4W	0.5L or 0.5S
5	1.0D + 1.0E	0.5L + 0.25S

There are several items to be noted about these load combinations:

a) The loads are all a companion action approach to all load cases so that the maximum factored portion of a load is evaluated with a smaller portion of the companion loads; hence, full factored snow and dead load is combined with 40% wind load.
b) Snow load is considered as a separate load case from live load so that the columns supporting both live and snow load will not have to resist the maximum of both at the same time.
c) For situations where dead load is much greater than live load there is a load case of 1.4 times dead load with no live load. This would occur in a column supporting several storeys of apartments where the live load reduction has reduced an already low live load and in tunnel roofs where there is large soil overburden.
d) NBCC contains a separate set of load combinations for use with cranes should there be a gantry crane in the building.

In general, those load cases that do not govern can be ignored, but there is an art in knowing which load cases are not significant. For example, wind loading is seldom an issue on simple span floor beams, and only the dead and live load combination is evaluated in those cases. If an element has components of load from several sources, then it is the responsibility of the designer to find the most severe load combination. Often very crude calculations can be performed to determine if a load case is significant enough to include.

For seismic loading (E) the loads are simplified (Clause 7.2.6) to

a) 1.0 D + 1.0E
b) 1.0 D + 1.0L + 1.0E for storage live load and assembly live loading
c) 1.0 D + 0.5L + 1.0E for other live loading

These are referred to as companion loads in that there is a main load condition and the loads likely to be present are also added in with factors of 1 or less. There is no addition of temperature, settlement, or wind effects to seismic loading. Under companion action load combinations for seismic loads, the seismic load is combined with unfactored dead load, and in most cases, 50% of the live load. Under the companion load approach we use 25% snow when calculating the seismic weight and 25% snow when calculating vertical effects occurring at the same time. The load combinations were changed under NBCC 2015; in particular, we are now required to combine 1.0S with factored live load and 1.0L with factored snow load. Previous editions had allowed a reduction for snow load when combined with factored live load. The reason for this change is to eliminate a potential underestimation of the design load when the ratio L/S is close to 1.0. We will show more the effect of combining snow and live load when computing the loading on a column that supports both roof load and floor load.

1.18 SERVICEABILITY LIMIT STATES

The serviceability limit is intended to reduce the number of complaints from users and owners of the buildings you design. While as structural engineers we spend much of our design time in providing a structure that is strong enough, it is the serviceability issues that will cause the greatest number of complaints. Visible deflections, bouncy floors, and roofs that deflect to the point that ceilings or roofing materials crack are all likely to result in complaints from the user and throw doubt on your structural design competence.

Serviceability deflections in steel are always computed using only the variable loads (live load, snow load or wind load). Permanent loads (dead load, superimposed dead load which includes finishes) are not used in the computation of deflections. The deflection for snow or wind have the serviceability limit state factor applied. For example, deflections under snow load where the SLS factor is 0.9 are computed under 0.9*Unfactored snow load. You might wonder why we generally don't combine snow and wind for computation of deflection of roof beams in Vancouver and the reason is that it is traditional not to do this. However, if we were designing a building in Saskatchewan where snow is long lasting and where snow and wind act in combination, then the designer should consider combining snow and wind for deflection checks. The consequences of too large of a deflection in a roof beam under combined snow and wind are not as serious as the consequences of a strength failure under combined snow and wind. Hence, we spend more effort on computing the strength resistance of the beam to combined wind and snow and in Vancouver; for example, we limit our concern with deflections to just looking at the individual cases of snow load or wind load.

It would probably be useful for the National Building Code to include a table for allowable stress combinations but allowable stress load combinations went away with NBCC 2005 to encourage all engineers including geotechnical engineers and those designing masonry, to perform their design using factored load provisions. The result of this is that there are no mandated load combinations for serviceability calculations and the decisions are more in the hands of the design engineer. The Building Code also does not provide limits on the deflections produced; these limits are left to the material standards. In the case of S16, the deflection limits are provided in non-mandatory Annex D . The limits are based on past performance of structures and different criteria are provided for industrial buildings and all other buildings. For vertical deflections in non-industrial buildings the limit is given as Span Length / 360. Deflections are computed for individual beams or girders. In a typically framed floor system with beams and girders, the deflection in the middle of the panel is not considered.

1.19 CAPACITY REDUCTION FACTORS

For structural steel, the capacity reduction factor Ø is given in Clause 13.1 as 0.9 unless otherwise specified. "Otherwise specified" would include the items shown in Table 1.13.

There are also a couple of other places in S16 where the resistance factor is changed and no symbol is given. Examples include the first compression diagonal of trusses and OWSJ where the resistance is multiplied by 0.85 to improve the post-yield behaviour.

Further information on load factors can be found in S16 Annex B (Margins of Safety).

Table 1.13. Resistance factors for various conditions

Item	Symbol	Value
Reinforcing Steel	$Ø_r$	0.85
Bolts	$Ø_b$	0.80
Shear Connectors	$Ø_{sc}$	0.80
Beam web bearing interior	$Ø_{bi}$	0.80
Beam web bearing exterior	$Ø_{be}$	0.75
Bearing of bolts on steel	$Ø_b$	0.80
Weld metal	$Ø_w$	0.67
Anchor Rods	$Ø_a$	0.67
Concrete	$Ø_c$	0.65

CHAPTER 2
LOADS

In most steel buildings several loads will be acting on the structure in both a vertical and lateral direction. While it is normal for dead and live loads to act together, we also need to think of less obvious combinations such as wind and snow loads.

2.1 DEAD LOADS

Dead load comprises the self weight of the structure itself along with the loads from components added during construction; for example, dead load from floor finishes, roofing, and partitions, all of which are typically shown on architectural drawings. Dead loads also include mechanical components such as sprinkler lines, electrical components such as lights and wiring, and rooftop mechanical equipment. Mechanical units on the roof are considered part of dead load and are added in to the snow load considering both to be present at the same time. The designer cannot consider that mechanical equipment replaces the snow load or, that when the equipment runs, its warmth will melt the snow. Drawings need to show the designer's assumptions for dead loads of both the structure and the superimposed dead load from partitions, finishes and mechanical and electrical components.

The use of "green roofs" in building design is also becoming more common, and soil and vegetation growth should be included as dead load so that their effects are combined with the snow load. The Building Code requires that shallow soil cover has a live load factor of 1.5 as there is a greater uncertainty in its depth and therefore magnitude than with most dead loads. Roofs with a deliberate growing medium are a reasonably new aspect of building design and it will be interesting to see how the weight of the green roof increases over time and if dust or other particulate matter in the air settle on the green roof, becoming part of the growing medium and increasing the weight on the roof.

2.2 LIVE LOADS

Drawings need to include a table showing the live loads used in design for the various parts of the building. Live loads are primarily vertical loads from the occupants of the building and the support of these loads is the reason that the building exists as shown in the example shown in Figure 2.1. Snow is not considered to be a live load but is a unique load case. Live loads are given in the Building Code and it is up to the design engineer to determine what the appropriate category should be. Most structural engineers spend very little time determining the appropriate live load to be used in design, preferring to devote the majority of time to designing elements that will resist this load. Aspects of choosing appropriate live load are discussed later in this chapter. Suffice it to say here that the Building Code requires designers to consider the effects of pattern live loading if leaving live load off one part of the structure increases the forces on another part of the structure. This is less of a challenge in steel design than it is in concrete design as steel beams are typically considered to have simple span connections and loading in one span does not affect

the adjacent spans. However if there are cantilever beams or other beams that are continuous over the support, then the effect of pattern loading must be considered with 100% live load and 0% live loads used.



Figure 2.1. Live load on a stadium stair at the end of a baseball game. Stairs and corridors are required to be designed for 4.8kPa

2.3 LIVE LOAD REDUCTIONS

When designing elements such as girders or columns that have a large tributary area, it is often permissible to reduce the live load according to formulas given in the Building Code. This load reduction is not permissible for snow load or assembly occupancy loading with a load less than 4.8kPa, eliminating live load reductions in schools or churches. Additional information on live load reduction can be found in the Building Code. If a column is supporting a large area of load subject to live load from *Use and Occupancy* as prescribed by the Building Code, there is a good probability that all of the floor area is not loaded to the maximum value called for in the Code. NBCC 2010 Clause 4.1.5.9 recognizes that this allows us to do a live load reduction if the tributary area satisfies minimum values depending on the type of loading. Code-permitted live load reductions are shown in Table 2.1.

Table 2.1. Code-permitted live load reductions

LOADING	REDUCTION
Snow	No Reduction
Assembly use LL < 4.8kPa	No Reduction
Assembly LL>4.8kPa (Area>80m2)	
Storage, Manufacturing, Retail, Garage, or Footbridge (Area>80m2) Where B is the tributary area of this occupancy supported by the member under review.	$0.5+\sqrt{\left(\frac{20}{A}\right)}$
Not Assembly or snow or otherwise covered above (Area>20m2). Where B is the tributary area of this occupancy supported by the member under review. (This category would include the minimum roof live load of 1.0kPa.)	$0.3+\sqrt{\left(\frac{9.8}{B}\right)}$

EXAMPLE—DESIGN A SIMPLE COLUMN

Find the live load reduction for a column supporting a second-floor office occupancy with a live load of 2.4kPa and a tributary area of 63m2.

The live load without reduction would be 2.4kPa * 63m² = 151.2kN. The office use occupancy permits us to use the live load factor of the bottom category of Table 2.1 and the live load factor can be computed as:

$$\text{LL Factor} = 0.3+\sqrt{\left(\frac{9.8}{B}\right)} = 0.3+\sqrt{\left(\frac{9.8}{63}\right)} = 0.694$$

The reduced live load tributary to the column for this occupancy at this level

Reduced Live Load = 0.694 * 151.2kN = 105kN

For column load takedowns it is often of more use to compute the live load reduction, this being the live load times one minus the live load factor.

LL Reduction = (1–0.694) * 151.2kN = 46.2kN

Note that if the column supports more than one use, each use must be evaluated separately. For example, if a column supports a roof, an office floor, and a parking level, then the roof snow load is not reduced. The office floor is reduced using the tributary area of office floor, and the parking level is reduced using the tributary area of parking.

2.4 POINT LOADS

The Building Code prescribes uniform loads that must be used when designing individual elements but also requires a separate consideration of point loads. NBCC requires the consideration of point loads that range from 1.3kN on an area of 200mm x 200mm (basically the weight of a worker standing on one foot) to 54kN over 250mm x 600mm for driveways over basements and areaways (occupied open spaces) that might represent the point load under one or possibly a pair of wheels at the end of the axle of a heavy vehicle or a jack being used to change a tire. While for the most part these point loads will not affect an individual beam (unless that beam is very short) it may have an influence on the design of the decking.

2.5 MINIMUM LIVE LOAD TO BE USED ON A ROOF

In the loads to apply for various uses and occupancies, roofs are given as 1.0kPa. This live load is intended to provide a minimum capacity for construction of the roof and to carry the weight of maintenance workers. It seldom governs except in the case of interior roofs that would occur on a kiosk in an atrium space or in roofs in regions with very low snow loads. The minimum roof live load requirement should be read in conjunction with NBCC, which states:

> roofs shall be designed for either the uniform *live loads* specified in Table 4.1.5.3, the concentrated *live loads* listed in Table 4.1.5.9, or the snow and rain loads prescribed in Subsection 4.1.6, whichever produces the most critical effects in the members concerned.

In other words, the live load from construction and maintenance activities need not be considered at the same time as the snow load. Under NBCC, snow is a separate load case distinct from live load, and some designers contend that the minimum live load should be considered when doing column takedowns by combining it with the companion action snow effects. This is not the intent of the minimum live load requirement and companion load. Instead the aim is as stated in NBCC, which requires that designers take the most critical loading when determining the axial load in a column. In the same way that the roof beams are not designed for snow load combined with the companion action effect of the minimum live load, the columns that support the roof are not subject to the companion action effect of the minimum live load combined with the snow. It should be noted for most columns the tributary area is large enough that the live load reduction can assist in reducing the minimum live load.

EXAMPLE—FIND MINIMUM LIVE LOAD

Find the effective minimum live load to be considered when designing a column supporting a roof occupancy with a live load of 1.0kPa and a tributary area of 63m².

The live load reduction falls into the bottom category shown in Table 2.1 and can be computed as:

$$0.3+\sqrt{\left(\frac{9.8}{B}\right)}$$

$$\text{LL Factor} = 0.3+\sqrt{\left(\frac{9.8}{B}\right)} = 0.3+\sqrt{\left(\frac{9.8}{63}\right)} = 0.694$$

The reduced live load tributary to the column for this occupancy at this level

Reduced Live Load = 0.694 * 1kPa = 0.69kPa.

The column supporting this roof loading this would **not** be combined with snow loading in the formula:

1.25D + 1.5L + 1.0S or 1.25D + 1.5S + 1.0L

In the same way that we do not combine the minimum live load as a companion load with snow, when we design the beams for the roof we do not need to combine the snow with minimum live load for the roof when we design the columns.

It is also important to remember that the Code is a minimum standard. If the designer knows there are mitigating factors and there is a probability of having some live load present at the time of the snow, then the design should reflect this. Consider the case of a car dealership where cars are stored on the roof. The cars are a live load, and with a strict interpretation of the Code, might not be added to the snow load on the roof. The dealer plans to store at least some cars on the roof at all times and some of the cars are likely to be on the roof at the time of the snow load. In this case the designer would be prudent to consider the cars as a storage load and include them with the snow load in both the design of the beams of the roof and the columns supporting that roof. This is especially true in those parts of Canada, including Greater Vancouver, where snow loading is the result of a single storm and not an accumulation from several storms.

EXAMPLE—DESIGN A SQUARE HSS COLUMN

Design a square HSS column supporting the roof of a warehouse type building located in Vancouver S_s = 1.8 and S_r = 0.2. The roof is essentially flat and has no snow drifting or ponding issues. The columns are spaced at 9m in one direction and 10m in the other direction. The overall size of the roof is 120m by 100m. Deck is 38mm with an allowance of 0.9kPa for roofing materials, ceiling, and suspended mechanical and electrical. The column supports a 530mm deep girder in one direction and 400 deep joists in the other direction.

Determine the appropriate snow loading for the building.

Roof size gives C_a = 1.00 (flat roof no projections or overhangs)

NBCC 2010 C_b:

$L_c = 2w - w^2/l = 2*100 - 100^2/120 = 117m$

$C_b = 1.0 - (30/L_c)^2 = 0.93$

Roof snow $= \{(C_b = 0.93) * (C_a = 1.00) * (C_s = 1.0) * (C_w = 1.0) * (S_s = 1.8)\} + (S_r = 0.2)$

$= 1.87kPa$

Check minimum live load.

Minimum live load = $1.0kPa * 0.3+\sqrt{\frac{9.8}{B}} = 1.0kPa * (0.3+\sqrt{\left(\frac{9.8}{10m * 9m}\right)})$

Minimum service live load = 1.0kPa * 0.63 = 0.63kPa

The minimum live load is less than the service snow load and will not be considered to be acting at the same time as the snow load (even as a companion live load).

2.6 CONCENTRATED LOADS AND THEIR EFFECT ON ROOF DESIGN

Under the Building Code we are required to design roofs for a concentrated load of 1.3kN spread over 750mm x 750mm to produce the maximum effects. A concentrated load of 1.3kN represents the weight of a worker with tools and will seldom govern unless there is a very short beam, and even then, it would be difficult to have either the beam and its connections not satisfy 1.3kN. As discussed above in Section 2.5 on minimum roof loading, the concentrated load is considered to act alone and need not be combined with either minimum uniform roof load or snow loads. If there are large mechanical units on the roof, then the loading for these should be considered as point loads, including the effect of any housekeeping pads.

2.7 DETERMINING THE SNOW LOADING ON THE ROOF

We will start our determination of the loads on roofs by determining the snow load for a flat roof which is determined from Clause 4.1.6.2 of the Building Code.

The basic roof snow load equation is: $S = I_s * (S_s * (C_b * C_w * C_s * C_b) + S_r)$

Is is the importance factor to be used with snow load. As with wind load there is an importance factor to be used with ultimate limit state (ULS) for strength design and another to be used with the serviceability limit state (SLS) for deflection design. The serviceability limit state factors for snow load are shown in Table 2.2.

Table 2.2. Serviceability limit state factors for snow load

BUILDING IMPORTANCE CATEGORY	ULS I_S (STRENGTH)	SLS I_S (DEFLECTION)
Normal Importance	1.0	0.9
High Importance	1.15	0.9
Post Disaster	1.25	0.9

S_s and S_r are the snow and rain parameters set by the building authority and under NBCC are based on a return period of 50 years. For Vancouver the values are given in the Vancouver Building Bylaw to be S_s = 1.8 and S_r = 0.2. (See current edition of Vancouver Building Bylaw.) For other areas the values of S_s and S_r are found in the Building Code; however, these values may be increased by the local jurisdiction, and when doing a project in a jurisdiction that the designer is not familiar with, it is worth checking the snow load with the local building department. The S_r and S_s values for one municipality can be quite different from that of its neighbours. Note that the values of S_s and S_r were computed from work done in the 1970s prior to any global warming effects. Recalculation of snow loads occurs with each Code.

C_b is the basic roof snow load multiplier. C_b = 0.8 for small roofs (maximum dimension less than 70m). Previous to NBCC 2005, C_b was always a constant of 0.8, which took account of the observed facts that the snow on the roof of a heated building is less than snow on the ground, partly because the snow

can drift off the roof. However under NBCC 2005, C_b increased to reflect the fact that with large roofs snow can drift from one location to another. For large roofs with Cw = 1 (see discussion on Cw below), the value of Cb will be:

$C_b = 1.0 - \left(\frac{30}{l_c}\right)^2$ (applies to roofs with l_c>70m)

l_c = characteristic length = $l_c = 2w - \frac{w^2}{L}$ where w = shorter roof dimension and l = longer roof dimension

C_s is the slope factor and is 1 for roof slopes less than 15 degrees regardless of the roof being slippery or not. Above this slope there are variations depending on whether the slope is classed as being slippery or not. For slippery roofs C_s = 0 at 60 degrees while for non-slippery roofs C_s = 0.0. When the roof slope exceeds 70 degrees, a value of C_s = 0 means that the Building Code considers that there is no snow on the roof. This is shown in Figure 2.2.

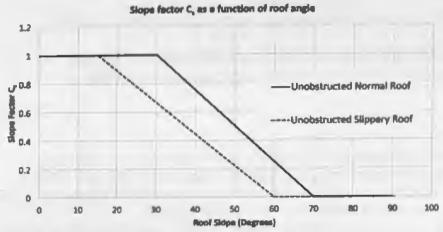


Figure 2.2. Slope factor C_s at various roof angles

C_w is a wind factor that should be taken as 1.0 in the Vancouver area. C_w may only be used to reduce snow load if the building is in open terrain on all sides and will remain so for the remainder of its life. C_w may not be used to reduce the snow load on *Post Disaster* buildings. C_w should also not be used in areas where there is not much wind in the winter, this includes some mountain valleys in British Columbia and some northern locations of the province where the wind loads are similarly low. In 1995 the Association of Professional Engineers and Geoscientists of British Columbia formed a committee on Part 4 of the Building Code comprised of consultants, academics, and building officials. One of the recommendations of that committee was a code interpretation as follows:

Code Interpretation: Sentence 4.1.*A*(1) and (3) for the effect of wind exposure factor C_w on roof snow loads:

Values of C_w less than 1.0 should normally not be used for evaluation of roof loads on buildings particularly in urban areas, and in coastal regions of British Columbia.

Reasons:

There is a possibility that obstructions higher than the roof may be constructed in the future resulting in a potentially unsafe condition.

Snowfalls in coastal areas of British Columbia are often very wet and occur under calm conditions. Reductions of roof snow loads due to wind are unlikely to occur.

The City of Vancouver currently does not permit the use of a C_w less tban 1.0.

EXAMPLE—FIND THE ROOF SNOW LOAD

Find the roof snow load for a normal importance building in Vancouver with a flat roof with dimensions 100m x 120m.

Determine the characteristic length

$$= l_c = 2w - \frac{w^2}{L} = 2(100m) - \frac{(100m)^2}{120m} = 116.7m$$

$$C_b = 1 - \left(\frac{30}{l_c}\right)^2 = 1 - \left(\frac{30}{117}\right)^2 = 0.93$$

$$S = I_s * (S_s * (C_b * C_w * C_s * C_a) + S_r = 1.0 * (1.8kPa * (0.93 * 1.0 * 1.0) + 0.2kPa) = 1.87kPa$$

EXAMPLE—FIND THE SNOW LOAD ON A SMALLER ROOF

Find the roof snow load for a normal importance building in Vancouver with a flat roof with dimensions 70m x 70m.

Determine the characteristic length $= l_c = 2w - \frac{w^2}{L} = 2(70m) - \frac{(70m)^2}{70m} = 70m$

$$C_b = 1 - \left(\frac{30}{l_c}\right)^2 = 1 - \left(\frac{30}{70}\right)^2 = 0.82 \text{ Say } 0.8$$

$$S = I_s * (S_s * (C_b * C_w * C_s * C_a) + S_r = 1.0 * (1.8kPa * (0.8 * 1.0 * 1.0) + 0.2kPa) = 1.64kPa$$

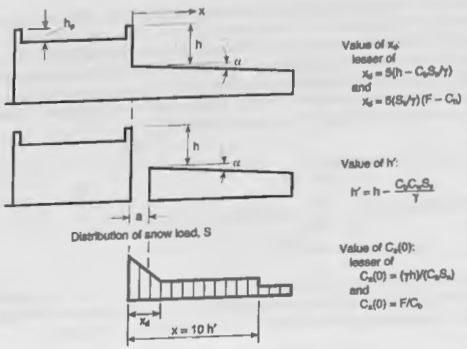
Note that 1.64kPa will be the flat roof snow load for all normal importance buildings less than 70m x 70m in Vancouver unless there are roof projections or steps causing snow build-up.

2.8 SNOW DRIFTING

Snow drifting can be a significant problem requiring changes to the framing such as the addition of beams or joists around roof projections such as penthouses to prevent overstress of the roof deck. While snow drifting in the Lower Mainland of British Columbia is often regarded as a phenomena that occurs only in climates colder and dryer than Vancouver the following should be kept in mind:

1) During a particularly bad snowstorm in December 1996, an aircraft hangar collapsed at Victoria airport even though it had been in existence for more than 50 years. Photographs taken in Victoria shortly after that storm clearly show drifting of snow had occurred in some locations.
2) During the Christmas snowstorm of 1997, drifting snow in Chilliwack was remarkably pronounced.
3) Even if drifting snow in Vancouver is a rare event, we are still required to design for rare events such as extreme winds and seismic events.
4) Many buildings are designed in Vancouver for areas where snow drifting does occur.
5) Designers are required by Code to design for snow drifting and the engineer of record will be sealing the drawings to state that they comply completely with the Code not just the parts of the Code the designer agrees with.

Figure 2.3 gives snow drifting adjacent to higher roof such as would occur adjacent to a mechanical penthouse or adjacent to a building setback. The figure covers a large number of snow drifting cases where a step in the roof occurs or where the roof is adjacent to a mechanical penthouse.



x	Factors[a]		
	C_w	C_s	C_a
0	1.0	$f(\alpha)^{[1]}$	$C_a(0)$
$0 < x \le x_d$	1.0	$f(\alpha)^{[x]}$	$C_a(0) - \frac{(C_a(0) - C_a(x_d)) \times}{x_d}$
$x_d < x \le 10\,h'$	1.0	$f(\alpha)^{[1]}$	1.0
> 10 h'	1.0[b]	$f(\alpha)^{[1]}$	1.0

Figure 2.3. Figure G-4 from Commentary G of the User's Guide – NBCC 2005 Structural Commentaries (Part 4). Similar figures are now included directly in the NBCC Code

Let's simplify Figure 2.3 further for one of the more common situations restricted to the following conditions:

1) Located in Vancouver or Lower Mainland or a sheltered area where $C_w = 1$.
2) Upper and lower roofs are essentially flat (<10 degree slope).
3) Upper and lower roofs have dimensions small enough to give $C_b = 0.8$ (*i.e.*, less than 70m on a side if square, longer if oblong).
4) Upper roof has a parapet that is small enough to ignore ($h_p = 0$).

The first equation to be solved is F.

$$F = 0.35 * \sqrt{\left(\frac{\gamma l_c}{S_s} - 6\left(\frac{\gamma h_p}{S_s}\right)^2\right)} + C_b \ge 2.0$$

where l_c = characteristic length of upper roof $= 2w - \frac{w^2}{l}$

C_b here refers to lower roof (take as 1.0 for roofs with dimension less than 80m)

h_p = height of parapet of upper roof.

γ = density of snow

The value of γ varies considerably depending on the geographical location, the elevation, the age of the snow, and the time of the season. NBCC Commentary G, item 6, states that snow at time of deposition (fresh snow) can be 0.5kN/m³ to 1.0kN/m³ and increases to 2.0kN/m³ to 5.0kN/m³. For the Lower Mainland a value of 3.0kN/m³ would be appropriate to use in the snow drift calculations while in drier areas at higher elevations a value closer to 1.0kN/m³ to 2.0kN/m³ would be more appropriate.

We find that F is dependent on the size of the roof but it is conservative to ignore the parapet [illegible] speed up hand calculations, resulting in the following equation:

$$\text{Use } F = 0.35 * \sqrt{\left(\frac{\gamma l_c}{S_s}\right)} + C_b \geq 2.0$$

Now: $C_a(0)$ is lesser of: $\frac{\gamma h}{C_b S_s} = \frac{\gamma h}{0.8 S_s}$ (small step cut-off)

or $\frac{F}{C_b} = \frac{F}{0.8}$ (C_b is 0.8 for roofs smaller than 70m)

The upper of the two equations only governs in small steps and it is conservative and quicker to ignore it in hand calculations. In the case of a 10m x 10m upper roof in Vancouver, the small step must about 1.2m or less to allow the small step equations govern, resulting in the following equations:

$$C_a = \frac{F}{0.8}$$

The snow at the step will be:

$$S = S_s * (C_b = 0.8) * (C_w = 1.0) * (C_s = 1.0) * (C_a = \frac{F}{0.8}) + S_r$$

Eliminating the terms that equal unity and simplifying get:

$S = (S_s * F) + S_r$

X_d is the lesser of: $5 * \left(h - \frac{C_b S_s}{\gamma}\right) = 5 * \left(h - \frac{0.8 * S_s}{\gamma}\right)$ (small step cut-off)

$$\text{or } 5 * \left(\frac{S_s}{\gamma}\right) * (F - C_b) = 5 * \left(\frac{S_s}{\gamma}\right) * (F - 0.8)$$

Again the upper equation governs only in the case of small steps and it is conservative and quicker to ignore this equation when doing hand calculations.

For Vancouver snow loads, the second equation will govern unless h is less than 1m to 2m. Again it is conservative to ignore the first equation. Using only

$X_d = 5 * \left(\frac{S_s}{\gamma}\right) * (F - 0.8)$ for hand calculations is quicker and is only slightly conservative for most reasonable steps, provided the following restrictions are met:

a) Located in Code mandated or wind sheltered areas where $C_w = 1.0$.
b) Upper and lower roofs are essentially flat (<10 degree slope).
c) Upper and lower roofs have dimensions small enough to give $C_b = 0.8$ (i.e., less than 70m on a side if square, longer if oblong).
d) Upper roof has a parapet that is small enough to ignore ($h_p = 0$).
e) Roof step is at least ½ metre. If the roof step is less than 1 metre, the results are conservative as the small step cut-off is not evaluated.

These simplifications result in the simplified loading diagram shown in Figure 2.4 as follows:

1) Compute characteristic length of upper roof = $l_c = 2w - \frac{w^2}{l}$ (w = short dimension of upper roof)

2) Compute $F = 0.35 * \sqrt{\left(\frac{\gamma l_c}{S_s} - 6\left(\frac{\gamma h_p}{S_s}\right)^2\right)} + C_b \geq 2.0$ use $\gamma = 3.0 kN/m^2$ if the snow is going to drift.

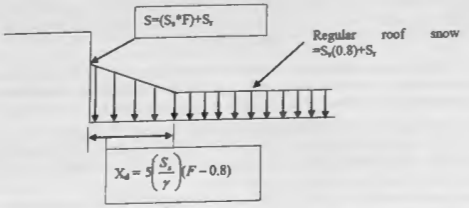


Figure 2.4. Simplified snowdrift loading diagram

EXAMPLE—DETERMINE SNOW DRIFT AROUND A PENTHOUSE

A 4m high mechanical penthouse sits on the 40m x 40m roof of a normal importance building ($I_s = 1.0$) to be located where $S_s = 1.8$, $S_r = 0.2$, and $C_w = 1.0$. The mechanical penthouse is 7m x 5m. Draw the snow load diagram adjacent to the penthouse.

Snow away from obstruction = 1.0 *(1.8 * 0.8 + 0.2) = 1.64kPa
Using our simplified assumptions for build-up area:

$$l_c = 2w - \frac{w^2}{l} = 2 * 5m - \frac{(5m)^2}{7m} = 6.4m$$

$$F = 0.35 * \sqrt{\left(\frac{\gamma l_c}{S_s} - 6\left(\frac{\gamma h_p}{S_s}\right)^2\right)} + C_b \geq 2.0$$

$$F = 0.35 * \sqrt{\frac{3.0 * 6.4}{1.8}} + 0.8 = 1.94 \text{ (Use minimum value of 2.0)}$$

$$C_a(0) = S_s * F + S_r = 1.8kPa * 2.00 + 0.2kPa = 3.80kN/m^2$$

$$X_d(0) = X_d = 5 * \left(\frac{S_s}{\gamma}\right) * (F - 0.8) = 5 * \left(\frac{1.8}{3}\right) * (2 - 0.8) = 3.6m$$

Result:
These simplifications result in the computed snow load diagram shown in Figure 2.5.

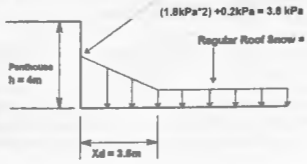


Figure 2.5. Computed snow load diagram

A computer program or spreadsheet to assist in calculating snow drift is of great value. Unfortunately such programs are not available commercially, so the designer will need to write it or work for a company that has a library of such programs.

2.9 MINIMUM ROOF LOADING

Under Building Code Table 4.1.5.3, Specified Uniformly Distributed Live loads on an Area of Flat Roof, the use and occupancy of roofs are listed with a load of 1.0kPa. We are directed to 4.1.5.1; however we should probably be directed to 4.1.5.5.2, which states:

> roofs shall be designed for either the uniform *live loads* specified in Table 4.1.5.3, the concentrated live loads listed in Table 4.1.5.10, or the snow and rain loads prescribed in Subsection 4.1.6, whichever produces the most critical effect on the members concerned.

The intent of the minimum roof load clause was to provide a live load allowance for workers on the roof and to provide minimum levels of rigidity. For most roofs subject to wind and snow loads in Canada, the minimum live load will usually not govern the design of the roof. The minimum roof loading of 1.0kPa will govern for interior roofs such as the roof over a retail unit in a high interior space such as an airport. The minimum roof live load of 1.0kPa has been taken into account, perhaps unintentionally, in the computation of column loads of multi-level buildings when using the companion action approach and we will discuss this further when we discuss loading on columns in Chapter 9.

2.10 WIND LOADING ON A BUILDING WITH A FLAT ROOF

Several points are worth keeping in mind when doing wind design:

1) The basic wind pressure is the value of the q_{50} value that is the wind pressure due to mean hourly wind with a period of greater than 50 years.
2) For Vancouver: $q_{50} = 0.48$kPa (Used for structural design for buildings and for cladding.)
3) The q_{50} pressures must be adjusted by the multiplying by several coefficients to give the pressures on the roof surface that are used in design. These factors include the importance factor, I_w, which depends on the importance classification of the building. The uplift pressure on the roof is a combination of interior pressure and exterior pressure and both must be calculated.

2.11 INTERIOR PRESSURE

The Building Code in Clause 4.1.7.1.3 gives the interior pressure as:

$$p_i = I_w * q * C_e * C_g * C_{pi}$$

I_w = Importance factor to be used for wind strength design and is shown in Table 2.3.

Table 2.3. Interior pressure

BUILDING IMPORTANCE CATEGORY	ULS I_W (STRENGTH)	SLS I_W (DEFLECTION)
Normal Importance	1.0	0.75
High Importance	1.15	0.75
Post Disaster	1.25	0.75

To find the values of C_{gi} & C_{pi} we have to go to Commentary I of the *User's Guide – NBCC Structural Commentaries (Part 4 of Division B)*. There is guidance as to the values of C_{gi} for internal pressure and the C_{pi} coefficient. If there are very few openings in the building, then it may be beneficial to compute C_{gi} using the procedure outlined in commentary Item 22, but the size and number of openings in the building skin has to be unreasonably small for this to be worthwhile and in most circumstances C_{gi} will be 2.0.

The exposure factor, C_e, for computing the effect of interior pressure is taken at the mid-height of the building. The interior pressure is a function of the background leakage of the skin of the building, possibly through openings for windows, vents, and doors. It is assumed that these are uniformly distributed over the height of the building and hence the wind speed of concern is the one at mid-height of the building. There is lots of opportunity for debate as to which category the building fits into. Some design offices consider that tilt-up warehouses fit into Category 1, but if all the loading dock doors are on one side, it would be fair to say that the background leakage would not be evenly distributed and this type of tilt-up building should fit into Category 2. There may also be future modifications to the building where a windowless building is modified to have a more storefront appearance resulting in increased background leakage.

Under NBCC provisions, the interior gust factor reflects the difference in pressure that occurs inside and outside the building. The wind pressures q_{50} are based on a one-hour average wind speed measured in open terrain at a height of 10m. The gust factor accounts for increases in pressures above the pressure derived from the one-hour average wind speed. Under NBCC provisions it is difficult to get a C_{gi} of less than 2.0 and Table 2.4 showing interior pressure coefficients has been modified to reflect this.

Table 2.4. Categories of interior pressure coefficients

CATEGORY	C_{pi}	C_{gi}	DESCRIPTION
Category 1	−0.15 to +0.0	2.0	Buildings without large openings and having small uniformly distributed openings. Applies to mechanically ventilated high-rises and windowless warehouses with door systems not prone to storm damage.
Category 2	−0.45 to +0.3	2.0	Buildings in which significant openings can be relied on to be closed but in which background leakage may not be uniformly distributed. Most low buildings fit into this category provided all elements (shipping door especially) are designed to be fully wind resistant.
Category 3	−0.7 to +0.7	2.0	Buildings in which large or significant exist which can transmit gusts to the interior including sheds and buildings where shipping doors and ventilators may be open in the storm.

Source: NBCC 2005 Commentary I-31

C_e the exposure coefficient is taken at mid-height of the building. For computing the internal pressure the value of the exposure factor for various heights is given in NBCC 4.1.7.1(5), resulting in the following formulas:

Open Terrain: $C_e = \left(\frac{h}{10}\right)^{0.3}$ but not less than 0.9

Rough Terrain: $C_e = 0.7 * \left(\frac{h}{12}\right)^{0.3}$ but not less than 0.7

Rough terrain is suburban or urban landscape or trees and is free of open areas, including large parking lots, large playing fields, open agricultural land, or bodies of water. The NBCC Structural Commentaries includes photos of rough terrain and open terrain and there will often be some engineering judgment will be involved. Rough terrain should not be assumed if the roughness is derived from trees that are to be cleared during the life of the structure. This may be difficult to predict for a structural engineer.

The rough terrain and exposed terrain conditions were brought in for NBCC 2005 as there were concerns that the higher wind loads for cladding would have a large financial impact. The feeling was that many buildings are in cities and large towns and would therefore benefit from the reduced wind load that can be realized from the rough terrain that exists there.

As with all wind loading there is plenty of scatter in the results. See also the NBCC Structural Commentaries for cases near hills and a transition formula where there are gaps in the upstream roughness.

For low buildings the C_e coefficients can be simplified as shown in Table 2.5.

Table 2.5. C_e coefficients for low buildings

Height (m)	Rough Terrain Exposure factor C_e	Open Terrain Exposure factor C_e
0 to 6m	0.7	0.9
>6m to 12m	0.7	1.0
>12m to 20m	0.8	1.1

It is important to keep track of the sign of the wind loads, as some will be trying to push the roof up while other pressure conditions will be trying to push the roof down. As shown in Figure 2.6 a positive external pressure will have a different effect than a positive internal pressure.



Figure 2.6. Positive external wind loads compared with positive internal wind loads

EXAMPLE—COMPUTE INTERIOR PRESSURE

Compute the interior pressure to be used for cladding and individual element design on an 18m high warehouse building in Vancouver that has several openings and is located in open terrain. The building is considered Category 2 as storm doors are designed to resist storms and are expected to be closed during a major wind storm but are not equally distributed.

Solution:

The basic equation for internal pressure is taken from NBCC 4.1.7.1(3) and is as follows:

Internal Pressure = $p_i = I_w * q * C_e * C_{gi} * C_{pi}$

$q = q_{50}$ used for all wind design under NBCC 2005 = 0.48kPa for Vancouver (these values can be obtained from NBCC Appendix C for communities throughout Canada). Cities such as Vancouver that have their own building code list the q_{50} value in Part 2 of the Code (see current edition of Vancouver building bylaw).

C_e = Exposure factor at mid-height = 18m / 2 = 9m gives $C_e = 1.0$

I_w = Importance factor to be used for wind strength design = 1.0 for normal importance

Determine Interior Building Pressure = $P_i = I_w * q * C_e * C_p * C_{pi}$ (Clause 4.1.7.1(3))

Height of building = 18.0m Reference Pressure q_{50} = 0.48kPa

Open Terrain:

$C_e = (H/10)^{0.2}$ but not less than 0.9:

C_e at mid-height of building = $(9m/10)^{0.2}$ = 0.98 (mid-height of building = 18m / 2 = 9m)

Building Category II – Normal Building Significant openings closed but not uniformly distributed

C_{pi} = 2.0 Range of C_{pi} = –0.45 to 0.3 (See NBCC Structural Commentaries, Part I)

Minimum Interior Building Pressure = 1.0 * 0.48 * 0.98 * 2.00 * –0.45 = –0.42kPa (– = Downward Load on Roof)

Maximum Interior Building Pressure = 1.0 * 0.48 * 0.98 * 2.00 * 0.30 = 0.28kPa (+ = Upward Load on Roof)

2.12 EXTERIOR PRESSURE

The Building Code in Clause 4.1.8.1.1 gives the exterior pressure as:

$$P = I_w * q * C_e * C_g * C_p$$

The pressures and gust factors to be used for most low-rise buildings are included in figures in Commentary I of the NBCC Structural Commentaries.

The exposure factor, C_e, for computing the effect of exterior pressure is taken at the level of the roof of concern. If the building has one roof level, take that level. If the building has two roof levels, then evaluate at each level separately. The effect of the wind gusts on the roof are a function of the wind speed at that level and hence the need to know the exposure factor appropriate for the roof level.

Figure 2.7 reproduces NBCC Structural Commentaries Figure I-9, External peak pressure coefficients, C_pC_g, on roofs with a slope of 7 degrees or less for the design of structural components and cladding. This figure uses q_{50} values for structural actions for both exterior and interior wind calculations. For the roof area, area r, and assuming a tributary area of 10m² or more this figure would give a maximum suction C_pC_g value of –1.5.

Determine Exterior Building Pressure = $P = I_w * q * C_e * (C_pC_g)$ (Clause 4.1.8.1.1)

Range of C_pC_g = –1.50 to 0.30 (Commentary Figure I-9)

C_e at roof of building = $(18m/10)^{0.2}$ = 1.12

Exterior Pressure at roof (– = up) = 1.0 * 0.48 * 1.12 * –1.50 = –0.81kPa

(– = upward load on roof)

Exterior pressure at roof (+ = down) = 1.0 * 0.48 * 1.12 * 0.30 = 0.16kPa

(+ = downward load on roof)

Some engineers will debate that Figure I-7 from the commentary should be used for the coefficients used in this calculation. The intent however is to use Figure I-7 when looking at the **complete** structure (for example, for determining wind overturning) and use Figure I-9 when looking at only an individual part of the structure in the case of wind girts or parts of the roof under wind pressure. Structural Commentaries Figure I-9 (cladding design pressures) should therefore be used for computing the wind uplift load on individual beams, joists, or girders.

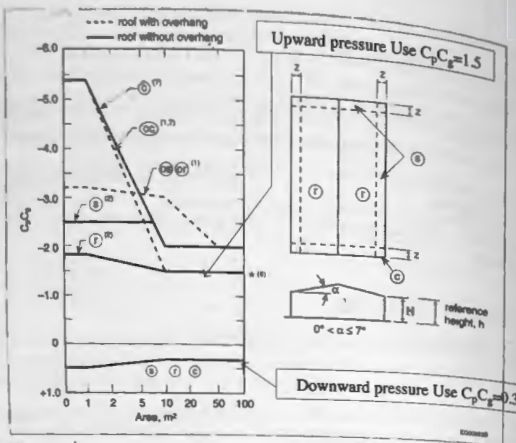


Figure I-9
External peak composite pressure-gust coefficients, C_pC_g, on roofs with a slope of 7° or less for the design of structural components and cladding

Figure 2.7. From Commentary I of the User's Guide – NBCC Structural Commentaries (Part 4) Figure on the right is Figure I-9 for the design of cladding and cladding support members the wind loading on a typical roof beam should use this figure

Care should be taken when determining the dead load that resists wind uplift. Most designers will use a conservative value of the roof dead load for design as this allows for some re-roofing late in the life of the building and future dead load but use of this load will not be conservative when computing resistance to wind uplift.

- As snow load helps to resist wind uplift, snow load is never considered in conjunction with wind uplift.
- Structural drawings should state design wind uplift used in design. This is particularly true when other designers will be working on the project. In particular, the Open Web Steel Joist designer will require this information. If wind uplift is present additional lines of bottom chord bracing will be required for the OWSJs.

EXAMPLE—NET FACTORED WIND UPLIFT

Determine the net factored wind uplift that should be used for joist uplift calculations on an 18m high building with a steel roof located in Vancouver in open terrain. The true weight of roofing, deck, and joists at time of construction is estimated to be 0.5kPa. The building is considered Category 2 for interior pressure as storm doors are designed for storm loading but are not evenly distributed.

Maximum Wind Pressures on Roof = $P_e - P_i$ (Clause 4.1.8.1.1)

From earlier parts of this example we have found:

The range of exterior pressure on the roof = –0.81kPa to +0.16kPa

The range of interior pressure = –0.42kPa to +0.28kPa

Maximum Suction Pressure on Roof = $P_e - P_i$ = –0.81 – (+0.28) = –1.09kPa (– = Upward load on Roof)

Maximum Downward Pressure on Roof = $P_e - P_i$ = 0.16 – (–0.42) = 0.58kPa (+ = Downward load on Roof)

Dead load of roofing and roof beam structure = 0.50kPa

Factored Net Uplift = 0.9D – 1.4W = (0.9 * 0.50) + (1.4 * –1.09) = –1.08kPa (– = Upward load on roof)

This would be the factored uplift that beams in the building would be designed to meet. A note should added to the general notes section of the structural drawings: "Open Web Steel Joists shall be designed to resist a net factored wind uplift of 1.1kPa."

Given the load combinations for use with NBCC discussed earlier, the loads for roofs require consideration of principal loads (dead load and snow load) and companion loads (wind loads).

Companion loads are only used with the principal loads if they make the situation worse in the case of wind uplift the 1.0S would not be used as it helps to counteract the wind force as snow cannot be relied on to be present when the wind is acting on the structure.

2.13 DOWNWARD WIND LOADING

For flat roofs the wind loading can exert a downward force on the roof. Prior to NBCC 2005 this was often ignored as the snow load was normally considerably larger than the downward load due to wind forces. The Code requires that wind be considered as a companion action to snow loading to the principal action dead load and snow. This gives the following load combination: 1.25D + 1.5S + 0.4W

EXAMPLE—DETERMINE THE SERVICE WIND DOWNWARD LOAD

Determine the service wind downward load that would occur on an 18m high normal importance building with a steel roof located in Vancouver in open terrain. The design weight of roofing, deck, and joists is estimated to be 0.5kPa. The building is considered a low-rise building and is Category 2 for interior pressure (access doors designed for wind load but not evenly distributed). Roof dimensions are 100m x 120m.

Maximum Downward Pressure on Roof = $P_e - P_i$ = 0.16 – (–0.42) = 0.58kPa (+ = Downward load on Roof)

Maximum Wind Pressures on Roof = $P_e - P_i$ (Clause 4.1.8.1.1)

From earlier parts of this example we have found:

The range of exterior pressure on the roof is: –0.81kPa to +0.16kPa

The range of interior pressure is: –0.42kPa to +0.28kPa

Maximum Suction Pressure on Roof = $P_e - P_i$ = –0.81 – (+0.28) = –1.09kPa (– = Upward load on Roof)

Maximum Downward Pressure on Roof = $P_e - P_i$ = 0.16 – (–0.42) = 0.58kPa (+ = Downward load on Roof)

Dead load of roofing and roof beam structure = 0.50kPa

Downward Load for Roof Design = 1.25D + 1.5S + 0.4W

Factored Downward Load = (1.25 * 0.50) + (1.5 * 1.88) + (0.4 * 0.58) = 0.63 + 2.82 + 0.23 = 3.68kPa (+ = Downward)

This would be the factored downward load that the joists would be designed to and a note should added the general notes section of the structural drawings: "Open Web Steel Joists shall be designed to resist a simultaneous downward wind load of 0.4*0.58kPa = 0.2kPa when computing factored loads due to snow."

For normal wind loading in Canada the downward wind loading increases the factored snow load ing combination by about 7% in the case of light roofs and less in the case of heavy roofs or heavy snow load (3.45kPa goes to 3.68kPa). Many structural designers in Vancouver might feel that this is unreasonable but the Building Code is a document intended to cover snow loading conditions throughout Canada where wind occurs more often and where the snow remains on the roof for longer periods of time thus increasing the probability of having snow on the roof when the wind occurs.

Figure 2.8 shows two load cases to consider depending on whether evaluating the beam for factored downward load or for the effects of wind uplift.

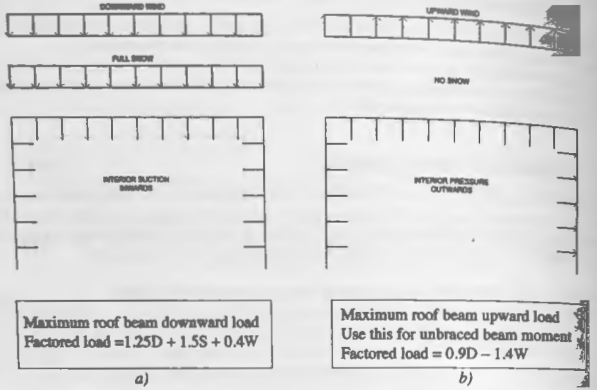


Figure 2.8. Maximum roof beam a) downward and b) upward load

EXAMPLE—DESIGN A ROOF BEAM FOR A NORMAL IMPORTANCE BUILDING

Design a roof beam for a normal importance building located in Vancouver in open terrain. Building is 18m high and is considered Category 2 as storm doors are designed for wind loading and will be closed during storms but are not uniformly distributed. Consider vertical snow load and wind forces. Design dead load from roofing, deck, and beams is 0.5kPa.The beams are spaced 2.5m apart and have an 8m span. The true weight of roofing, joists, and deck at time of construction is estimated to be 0.5kPa. Roof dimensions are 100m x 120m.

Factored dead and snow and wind loading:

Factored Downward Load = (1.25 * 0.50) + (1.5 * 1.88) + (0.4 * 0.58) = 0.63 + 2.82 + 0.23 = 3.68kPa (+ = Downward)

Load on beam = 3.68kPa * 2.5m = 9.2kN/m

$$\text{Factored D+L Moment} = M_f = \frac{9.2kN/m*(8m)^2}{8} = 73.6kNm$$

The factored net wind uplift is computed from the previous example as 1.08kPa

Factored Net Uplift = 0.9D – 1.4W = (0.9 * 0.50) + (1.4 * –1.09) = –1.08kPa

(– = Upward load on roof)

Factored uplift = w_f = {1.08kPa}* 2.5m = 2.7kN/m

$$\text{Factored Uplift Moment} = M_f = \frac{2.7kN/m*(8m)^2}{8} = 21.6kNm$$

(Note that the factored uplift moment is only 30% of the factored dead and live moment; nevertheless it is significant as the compression flange is unsupported.)

Span / depth = 25 Depth = 8000 / 25 = 320 ➔ Try W310.
W310x21 has a M_r of 89.1kNm (OK satisfies factored D+L)

at 8.0m using ω_2 = 1.13 M_r' is 8.49*1.13 = 9.6kNm (Not OK does not satisfy wind uplift).

Have to go to W310x39 section before we get an M_r' of greater than 25kNm.

(M_r' of W310x39 = 51.8kNm) A W410x39 would also be appropriate with M_r' of 36.6kNm.

Note that W310 x31 would work but this section is in light type (unique Canadian section no longer rolled so go to next size that works).

Check deflection using W310x39

$$\text{LL deflection} = \Delta = \frac{5*w_l*L^4}{384*E*I} = \frac{5*(1.88kPa*0.9*2.5m)*8.0^4*1000}{384*200*85.1} = 13\,mm = L/603\,(\text{OK})$$

Minimum I to satisfy deflection criteria = (360 / 603) * 85.1 x 10^6mm^4 = 50.8 x 10^6mm^4

Note that there are no sections with less depth and weight than the W310x39 that meet the three criteria:

- $M_r \geq 73.6$kNm
- $M_r'(8m) \geq 21.6$kNm
- $I \geq 50.8 \times 10^6$mm^4

Use W310x39 size controlled by wind uplift.

Of course, not all roof beams are governed by wind uplift especially as the weight of the roofing gets to a more reasonable weight allowance of 1.0kPa plus decking, beams, and suspended mechanical for a total of 1.4kPa. Let's look at a small roof in a built-up area of Vancouver that can be considered to be rough terrain. We will look at a beam with a span of 8.0m and with a tributary width of 2.75m.

EXAMPLE—DESIGN A ROOF BEAM FOR A COMMERCIAL BUILDING

Design a roof beam for a normal importance commercial building located in Vancouver (S_s = 1.8, S_r = 0.2, q_{50} = 0.48kPa) in rough terrain. Building is 10m high and is considered Category 2 as storm doors are designed for wind loading and will be closed during storms but not uniformly distributed. Consider vertical snow load and wind forces. Design dead load from roofing, deck, and beams is 1.4kPa and the beams are spaced at 2.75m apart and have an 8.0m span. Roof dimensions are 30m x 40m. There are no ponding or snow drift issues.

Basic roof snow according to NBCC 2005 / BCBC 2006 Clause 4.1.6.2(1) to (8)

Ground Snow Properties: S_s = 1.8 S_r = 0.2

Dimensions in metres of roof: Length = 40.0m Width = 30.0m

Building is Normal Importance. Importance factors: Snow $I_s = 1.00$ Wind $I_w = 1.00$

$C_w = 1.0$ Cs = 1.0 Roof is low slope and wind does not blow snow off roof

Compute characteristic length and basic snow for roof

$L_c = 2*W-(W*W/L) = 2 * 30.0 - (30.0*30.0 / 40.0) = 37.5m$

Characteristic Length of Roof = 37.5m

C_b for roof using $C_w = 1$ formula $C_b = 1-((30/L_c)^2) = 1 -((30 /37.5)^2) = 0.36$

Minimum value of 0.8 shall be used for C_b of roof

Snow load on roof = $I_s*(S_s*C_b*C_w*C_s*C_s + S_r)$

Basic Snow load on roof = 1.00 * (1.8 * 0.80 * 1.0 * 1.0 * 1.0) + 0.2 = 1.64kPa

Determine Interior Building Pressure = $P_i = I_w * q * C_e * C_p * C_{pi}$ (Clause 4.1.7.1(3))

Height of building = 10.0m Reference Pressure q_{50} = 0.48kPa

Rough Terrain: $C_e = 0.7*(H/10)^{0.7}$ but not less than 0.7: C_e at mid-height of building = 0.76 Roof = 0.70

Building Category II – Normal Building Significant openings closed but not uniformly distributed

C_{pi} = 2.0 Range of C_{pi} = –0.5 to 0.3 (See NBCC Commentary Part I)

Minimum Interior Building Pressure = 1.0 * 0.48 * 0.70 * 2.00 * –0.45 = –0.30kPa (– = Downward Load on Roof)

Maximum Interior Building Pressure = 1.0 * 0.48 * 0.70 * 2.00 * 0.30 = 0.20kPa (+ = Upward Load on Roof)

Determine Exterior Building Pressure = $P = I_w * q * C_e * (C_pC_g)$ (Clause 4.1.8.1.1)

Range of C_pC_g = –1.50 to 0.30 (Commentary Figure I-9)

Exterior Pressure at roof (– = upward) = 1.0 * 0.48 * 0.70 * –1.50 = –0.50kPa (– = Upward load on roof)

Exterior Pressure at roof (+ = Downward) = 1.0 * 0.48 * 0.70 * 0.30 = 0.10kPa (+ = Downward load on roof)

Maximum Wind Pressures on Roof = $P_e - P_i$ (Clause 4.1.8.1.1)

Maximum Suction Pressure on Roof = $P_e - P_i$ = –0.50 – (+0.20) = –0.71kPa (– = Upward load on Roof)

Maximum Downward Pressure on Roof = $P_e - P_i$ = 0.10 – (–0.30) = 0.40kPa (+ = Downward load on Roof)

Dead load of roofing and roof beam structure = 1.40kPa

Factored Net Uplift = 0.9D – 1.4W = 0.9 * 1.40 + 1.4 * –0.71 = 0.27kPa (+ = Downward load on roof)

As the above result is positive, no net uplift on roof and uplift design are required

Downward Load for Roof Design = 1.25D + 1.5S + 0.4W

Downward Load = (1.25 * 1.40) + (1.5 * 1.64) + (0.4 * 0.40) = 1.75 + 2.46 + 0.16 = 4.37kPa (+ = Downward)

Use above Values for Beam Design: Trial Beam: W310x24

Beam Span = 8.0m Beam Tributary width = 2.75m

Maximum positive moment on beam = 4.37kPa * 2.75m * (8.00m)² / 8 = 96.2kNm

Beam F_y = 345 Beam Class = 1 $M_r = Ø*F_y*Z_x = 0.90 * 345.0 * 328.0 *0.001$ = 102kNm

Beam Moment Resistance of 101.8kNm exceeds maximum positive moment of 96.2kNm OK

No net uplift. Do not need to check for negative moment bending.

Compute Deflection using 0.9 * basic snow load: 0.9 * 1.64 = 1.48kPa

Beam Load = (0.9 / 1.00) * 1.64 * 2.75 = 4.06kN/m

W310x24 Moment of Inertia = 42.7 x10^6mm^4

Deflection = 5 * 0.9 * 1.64 * 2.75 * 8.000^4 / (384 * 200 * 42.7) = 0.025m

Service Load Deflection on beam = 25.3mm = L / 316 **NO GO**

The required moment of inertia of the beam is (360/316) * 42.7 = 48.6 x10^6mm^4

Go to W310x28 M_r = 126kNm (OK > 96.2kNm)

I = 54.3 x 10^6mm^4 (OK > 48.6 x 10^6mm^4)

EXAMPLE—DESIGN A ROOF BEAM FOR A FIREHALL

For a firehall in Chilliwack, find the roof loading parameters and size a beam to span 7m and have a tributary width of 2.5m. The building is 9m high and 25m x 35m in plan and the fire bay doors may be opened under windy conditions. The q_{50} value is 0.72kPa, snow coefficients are S_s = 2.2kPa and S_r = 0.3kPa. As a fire hall the building is a post-disaster facility. Dead load of beams, deck roofing, and suspended electrical and mechanical is 1.35kPa. Try W310x24 at first. Maximum deflection permitted is L/360.

Loading calculations:

Basic roof snow according to NBCC 2005 / BCBC 2006 Clause 4.1.6.2(1) to (8)

Ground Snow Properties: S_s = 2.2 S_r = 0.3

Dimensions in meters of roof: Length = 35.0m Width = 25.0m

Building is Post Disaster Importance factors: Snow I_s = 1.25 Wind I_w = 1.25

C_w = 1.0 Cs = 1.0 Roof is low slope and wind does not blow snow off roof

Compute characteristic length and basic snow for roof

$L_c = 2*W-(W*W/L) = 2 * 25.0 - (25.0*25.0 / 35.0) = 32.1m$

Characteristic Length of Roof = 32.1m

C_b for roof using $C_w = 1$ formula $C_b = 1-((30/L_c)^2) = 1 -((30 /32.1)^2) = 0.13$

Minimum value of 0.8 shall be used for C_b of roof

Snow load on roof = $I_s*(S_s*C_b*C_w*C_s*C_s + S_r)$

Basic Snow load on roof = 1.25 * (2.2 * 0.80 * 1.0 * 1.0 * 1.0) + 0.3 = 2.58kPa

Determine Interior Building Pressure = $P_i = I_w * q * C_e * C_p * C_{pi}$ (Clause 4.1.7.1(3))

Height of building = 9.0m Reference Pressure q_{50} = 0.72kPa

Open Terrain: $C_e = (H/10)^{0.2}$ but not less than 0.9: C_e at mid-height of building = 0.90
C_e at Roof = 0.98

Building Category III – Large or significant openings in building, including open-sided sheds

C_{pi} = 2.0 Range of C_{pi} = –0.7 to 0.7 (See NBCC Commentary Part I-22)

Minimum Interior Building Pressure = 1.3 * 0.72 * 0.90 * 2.00 * –0.70 = –1.13kPa (– = Downward Load on Roof)

Maximum Interior Building Pressure = 1.3 * 0.72 * 0.90 * 2.00 * 0.70 = 1.13kPa (+ = Upward Load on Roof)

Determine Exterior Building Pressure = $P = I_w * q * C_e * (C_pC_g)$ (Clause 4.1.8.1.1)

Range of C_pC_g = –1.50 to 0.30 (Commentary Figure I-9)

Exterior Pressure at roof (– = upward) = 1.3 * 0.72 * 0.98 * –1.50 = –1.32kPa (– = Upward load on roof)

Exterior Pressure at roof (+ = Downward) = 1.3 * 0.72 * 0.98 * 0.30 = 0.26kPa (+ = Downward load on roof)

Maximum Wind Pressures on Roof = $P_e – P_i$ (Clause 4.1.8.1.1)

Maximum Suction Pressure on Roof = $P_e – P_i$ = –1.32 – (+1.13) = –2.46kPa(– = Upward load on Roof)

Maximum Downward Pressure on Roof = $P_e – P_i$ = 0.26 – (–1.13) = 1.40kPa (+ = Downward load on Roof)

Dead load of roofing and roof beam structure = 1.35kPa

Factored Net Uplift = 0.9D–1.4W = 0.9 *1.35 + 1.4 * –2.46 = –2.22kPa (– = Upward load on roof)

Downward Load for Roof Design = 1.25D + 1.5S + 0.4W

Factored Downward Load = (1.25 *1.35) + (1.5 * 2.58) + (0.4 * 1.40) = 1.69 + 3.86 + 0.56 = 6.11kPa (+ = Downward)

Loading Results

Factored Net Uplift = (0.9 * Dead) – (1.4 * Up Wind) = –2.22kPa

Factored Downward Load = (1.25 * Dead) + (1.5 * Snow) + (0.4 * Down Wind) = 6.11kPa

Now check selected beam: Trial Beam: W310x24

Check Positive moment (tension in bottom flange)

Beam Span = 7.0m. Beam tributary width = 2.5m

Maximum positive moment on beam = 6.11kPa * 2.50m * $(7.0m)^2$ / 8 = 93.5kNm

Beam F_y = 345 Beam Class = 1 $M_r = Ø*F_y*Z_x$ = 0.90 * 345.0 * 328.0 *0.001
= 102kNm

Beam Moment Resistance of 101.8kNm exceeds maximum positive moment of 93.5kNm OK

Check Negative moment (unsupported bottom flange in compression)

Maximum negative moment on beam = 2.50m * –2.22kN/m * $(7.0m)^2$ / 8 = –34.0kNm

Compute M_r' using ω_2 = 1.13 Beam unsupported length = 7.00m

M_r' = 14.43kNm

Beam Moment Resistance of 14.4kNm is exceeded by maximum negative moment of –34.0kNm
NO GO

Check Deflection using service snow loading:

Compute Deflection using 0.9 * basic snow load: (0.9 / 1.25) * 2.58 = 1.86kPa

Original Snow Importance factor of: 1.25 has been removed from above equation

Beam Load = 1.86kPa * 2.50m = 4.64kN/m

W310x24 Beam Inertia = 42.7 x 10^6mm^4

Deflection = 5 * 4.64kN/m * $(7.0m)^4$ / (384 * 200 * 42.7) = 0.017m

Service Load Deflection on beam = 17.0mm = L / 413 OK

Result:
While our proposed W310x24 works for positive moment and deflection, it does not have sufficient capacity for uplift wind loads with an unsupported compression flange distance of 7.0m. We will now try for a beam that has a M_r' of 34.0 / 1.13 = 30.0kNm in the beam selection tables (blue pages) of the *CISC Handbook* with ω_2 of 1.00. Try W250x33.

Check Positive moment (tension in bottom flange)

Beam Span = 7.0m Beam Tributary width = 2.5m

Maximum positive moment on beam = 6.11 * 2.50m * $(7.0m)^2$ / 8 = 93.5kNm

Beam F_y = 345 Beam Class = 2 $M_r = Ø*F_y*Z_x$ = 0.90 * 345.0 * 424.0 *0.001 = 132kNm

Beam Moment Resistance of 131.7kNm exceeds maximum positive moment of 93.5kNm OK

Check Negative moment (unsupported bottom flange in compression)

Maximum negative moment on beam = 2.50m * –2.22kPa * $(7.0m)^2$ / 8 = –34.0kNm

Compute M_r' using ω_2 = 1.13 Beam unsupported length = 7.00m

M_r' = 45.56kNm

Beam Moment Resistance of 45.6kNm exceeds maximum negative moment of –34.0kNm OK

Check Deflection using service snow loading:

Compute Deflection using 0.9 * basic snow load: (0.9 / 1.25) * 2.58 = 1.86kPa

Original Snow Importance factor of 1.25 removed from above equation

Beam Load = 1.86kPa * 2.50m = 4.64kN/m

W250x33 Beam Inertia = 48.9 x 10^6mm^4

Deflection = 5 * 0.9 * 2.58 * 2.50 * 7.000^4 / (384 * 200 * 48.9) = 0.015m

Service Load Deflection on beam = 14.8mm = L / 472 OK

Result:
The W250x33 works for all three criteria: (1) supported compression flange positive moment, (2) unsupported compression flange negative moment, and (3) deflection under service snow load. A review of the capacities and beam inertias given in the beam selection tables (blue pages) of the *CISC Handbook* will show that the W250x33 is the lightest beam that will meet all three criteria.

2.14 PONDING

Ponding is the excessive accumulation of water at low-lying areas of a roof. Ponding can lead to failure of a roof in a sudden and dramatic manner, as shown in Figure 2.9.

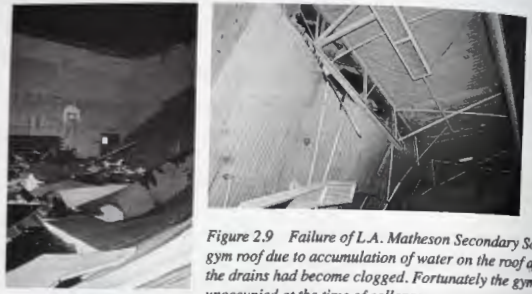


Figure 2.9 Failure of L.A. Matheson Secondary School gym roof due to accumulation of water on the roof after the drains had become clogged. Fortunately the gym was unoccupied at the time of collapse

2.15 DETERMINING THE LOAD TO BE USED IN DESIGN

The loads to be used by the designer are specified by the Building Code Table 4.1.5.3 but often judgment calls need to be made about which loading to use. For example, a room in a below ground level at a public library that is used for meetings and seminars could fit into any one of the following categories found in Table 4.1.5.3:

- Assembly areas: Auditoria 4.8kPa
- Assembly areas: Classrooms with or without fixed seats 2.4kPa
- Libraries: Stacks 7.2kPa
- Libraries: Reading and study rooms 2.9kPa

The designer wishes to satisfy Code requirements but also wishes to make the design economical.

It should be noted that many of these loadings have historic origins and are often very conservative when evaluated on a day-to-day basis. Assuming the average dressed person with normal accessories weighs 200 lbs (0.9kN) (see Table 2.6 for Transport Canada airline regulations related to calculating passenger weight) and assuming that each person will feel very uncomfortable with a space of less than 1m x 1m we get a maximum load of 0.9kPa.

Table 2.6. Segmented weights for passengers aged 12 years and older (in pounds)

MAXIMUM CERTIFICATED PASSENGER SEATING CAPACITY	WINTER		SUMMER	
	FEMALES	MALES	FEMALES	MALES
1–4	Use actual weights, volunteered weights, or estimated weights			
5	202	242	196	236
6–8	198	237	192	231
9–11	191	229	185	223
12–16	188	226	182	220
17–25	185	222	179	216

Source: Transport Canada

Some loadings are much more likely to be achieved in practice; for example, offices above the ground floor that may under present Code provisions be designed for 2.4kPa live load will often be exceeded in "normal" filing areas that are present in most offices. Particular attention should be made to compact file storage areas when reviewing buildings that have been designed to low uniform live loads as areas of rolling file storage measured loading can go as high as 14kPa (300 Psf).

Snow and wind loading values are given in Appendix C for locations in Canada based on a return period of 1 in 50 years. This means that in any one year there is a probability of exceedance of $1/50 = 0.02$ or 2%. The probability that the load given in the supplement will not be exceeded in any one year is therefore $(1–0.02 = 0.98)$ or 98%. For series of n years the probability of not being exceeded can be defined as $(1–0.02)^n$. If we expect a 75-year life on our building, there is therefore a probability of $(1–0.02)^{75} = 0.22$ or 22% probability that the snow or wind load will *not* be exceeded. In other words there is a 78% chance that it will be exceeded. The foregoing discussion assumes that the snow and wind loads are independent from one year to the next and the loads given in Appendix C are correct. Unfortunately most of the values in Appendix C were derived from studies done in the 1960s and 1970s and predate any influences of global warming.

Environmental loads given in the Code are not the maximum credible load on the structure. Instead, the Building Code defines a return period that is deemed to provide an expected level of safety. There is a very good chance that this load will be exceeded during the life of the structure. For example, take Table 2.7 for wind, which shows that there is an 87% probability of exceedance for the Code-defined q_{50} wind load in 100 years. Similarly, the Code says we are to design buildings for a 1 in 50 year snow load.

Table 2.7. Wind load and probability of being exceeded

Wind load	Vancouver Code specified reference pressures (kPa)	Probability of exceedance per year	Probability of being exceeded in 50 years	Probability of being exceeded in 75 years	Probability of being exceeded in 100 years
q10	0.36	10.0%	99.5%	100.0%	100.0%
q30	0.44	3.3%	81.6%	92.1%	96.6%
q50	0.48	2.0%	63.6%	78.0%	86.7%
q100	0.53	1.0%	39.5%	52.9%	63.4%

The purpose of the load factor is to take account of overloads that occur beyond the Code-mandated load.

We will discuss the return periods for seismic loading in Chapter 14 during our introduction to seismic design.

2.16 CONSIDER USING A MINIMUM FORCE AND AVOIDING DAMAGE

When considering small items of framing, the designer should consider the effect of vandalism or accidental loading during construction and use and apply a point load of 1kN service (1.5kN factored) at any point on the structure. This would apply to miscellaneous metal for signs and structural art pieces. In some structural engineering firms that design industrial structures, a 20kN (5 Kip) (service) point load at any location is a design requirement.

2.17 LOAD SHEETS

One way to make the design notes and calculations easier to produce and follow later is to eliminate repeating calculations by reference back to calculations that are common to several elements. An example of this is the preparation of load sheets to provide the design loads for specific areas of the project. Using load sheets it is very quick to determine the live load and factored loading for an element such as a beam or a column by multiplying the loading from the load sheet by the tributary area for the element. There are several ways to produce load sheets, but one of the most effective methods is to use a spreadsheet approach and summarize the loading from each source and compute subtotals that apply to the deck, beams, and girders. An example of this type of load sheet is shown in Table 2.8. The dead load and live load are kept separately, as it will be necessary to have both the factored loading and the live loading when designing elements such as beams, where both forces due to factored loads and deflections due to service live loads are checked. The load sheets show very clearly what items have been included in the design of the structural system. It is worthwhile considering carefully if the loading is reasonable and if all items have been considered. Ignoring the weight of housekeeping pads on mechanical floors or the weight of a raised floor can make it difficult to rationalize the safety of the structure to carry these loads later on. Often too much of the design effort is devoted to designing the resistance of the system without sufficient consideration of the loads that are present.

The beam self-weight is added directly in the load sheet to alleviate the need to add it in separately to each beam calculation. This procedure supposes that the self-weight of the beams is estimated using a trial design or on the basis of experience with similar designs. If desired, the estimate of beam weight can be checked following the design of several elements, with the calculations for these elements modified if necessary. However, the beam self-weight is usually only a small percentage of the total load carried by the beam, and as long as some estimate of beam self-weight has been included in the beam design, then the variations in the beam self-weight will usually not result in beam size changes. Beam self-weight will be a more significant portion of the total beam loading where spans are longer or in cases where the superimposed dead and live load are low.

Table 2.8. Classroom block floor loading (except corridors)

LOAD SOURCE	DEAD (kPa)	LIVE (kPa)	FACTORED (kPa)
Classroom loading (Code Table 4.1.5.3)		2.4	3.6
Partition load / finishes / suspended mech. & elect.	1.0		1.3
65 topping on 38 deck	2.0		2.5
Total load for Deck Design	3.0	2.4	7.4
W410 beam at 2500 o/c	0.2		0.2
Total load for Beam Design	3.2	2.4	7.6
W610 beam at 8000 o/c	0.1		0.1
Total load for Girder Design	3.3	2.4	7.7

In addition to being helpful when designing for vertical loading, the load sheets can be used for determining the weight of the building for seismic analysis.

2.18 DESIGN OF ROOF BEAMS

The design of roof beams is more complex than for floor beams as there are three criteria to satisfy:

1) Maximum factored resistance of the beam must be greater than the factored positive moment on the beam. The factored positive moment puts compression at the top flange, which is usually laterally restrained by the roof deck. As we will see, the load on the beam is a combination of dead load, snow load, and downward wind effects.
2) Deflection of the beam under "service" snow load conditions.
3) Maximum factored resistance of the beam under upward wind. This produces a negative moment on the beam and results in the unsupported bottom flange of the beam being in compression. The factored negative moment on the roof beam must be less than the M_r' resistance of the beam, with its compression flange unsupported over its full length. While the wind uplift load may be less than half of the factored gravity load, the unsupported length of the beam has often reduced the capacity of the beam to less than half its supported capacity, and this criterion will often govern the design of roof beams.

To design roof beams, we need to demonstrate how to find the snow and wind loads on the roof. We will do this for the most typical case of steel roof, which is a flat roof.

EXAMPLE—DETERMINE THE LOAD ON THE CENTRAL BEAM

A 20m x 24m roof is illustrated in Figure 2.10. The roof is located in Vancouver ($S_s = 1.8$, $S_r = 0.2$, one-day rain = 112mm) and its use is retail. The edge of the roof is at a constant elevation, but to provide slopes, the tops of the two columns are 200mm below the roof at the edge. There is an 8-inch up-stand parapet at the roof edge. There are scuppers but they are 3 inches above the roof edge. Determine the load on the central beam (Beam A) from snow loading and ponding.

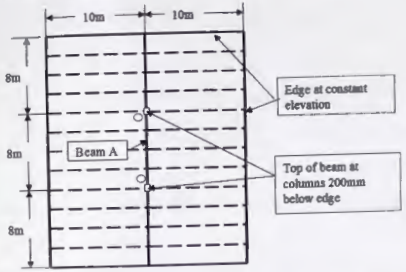


Figure 2.10. Central beam snow loading and ponding

Solution:

The environmental properties are as follows:

S_s = 1.8kPa S_r = 0.2kPa One-day rain = 112mm

The building is retail use and is therefore considered to be Normal Importance.

The size is such that the snow accumulation factor C_b = 0.8 and there are no projections or slopes such that C_a = 1.0

Snow:

$S = I_s * (S_s*(C_b * C_w * C_s * C_a) + S_r = 1.0*(1.8kPa*(0.8 * 1.0.* 1.00) + 0.2kPa) = 1.64kPa$

Beam A has a tributary width of 10m and therefore the loading is:

Snow Load: 10m * 1.64kPa = 16.4kN/m

Ponding:

Determine the volume of rain that falls on the roof in one day = 0.112m * 20m * 24m = 53.8m³

This volume of water must be retained on the roof. The volume will be contained in two portions: (1) the sloping portion up to the edge line will fill first, and (2) the remainder will go into a 20m x 24m rectangle on top of this. The problem is made manageable by assuming as an initial approach that the beams and the joists do not deflect.

Determine the volume of rain that is contained in the sloping portion of the roof (i.e., that portion is between the valley of the roof and the bottom of the parapet).

Each of the four end corners (8m x 10m in plan) volume contains

$$V = \left(\frac{Width * (MaximumDepth / 2)}{2}\right) * Length = \frac{10m * (0.2m / 2)}{2} * 8m = 4m^3$$

The central volume contains

$$V = \left(\frac{Width * (MaximumDepth)}{2}\right) * Length = \frac{10m * 2 * (0.2m)}{2} * 8m = 16m^3$$

The total volume in the undeflected roof sloping portion up to the edge line is

$$V = 4 * (4.0m^3) + 16m^3 = 32m^3$$

The remaining volume is the volume of one-day rain less the volume contained in the sloping portion:

$$V = 53.8m^3 - 32m^3 = 21.8m^3$$

The height of the water in the sloping portion is $21.8m^3/(20m*24m) = 0.045m = 45mm$ (Note that this is less than the 75mm of the scuppers plus the 30mm overflow required by 4.1.6.4(4) for a total of 105mm – one-day rain is retained on the roof and does not escape from the scuppers.) Thus 45mm will be the depth of water at the parapet.

The total depth over the beam is therefore 0.200m + 0.045m = 0.245m

Now take approximate account of the deflection of the beam and the resulting extra depth of water over the beam.

Account for L/360 deflection of the beam = 8000mm / 360 = 22mm

(A lengthier and less conservative approach can be taken by accounting for the actual beam inertia and resulting deflection, but the approach proposed here is reasonable and quick.)

Total depth of water directly over the beam = 245mm + 22mm = 267mm

Depth of water at the parapet = 45mm

At the centre of the roof the water depth is 267mm, giving a load of 2.67kPa (about 60% higher than the snow load), while at the edges the water depth is 45mm with a load of 0.45kPa. By doing a free body calculation of the joists it can be shown than the live load on the central beam is shown in Figure 2.11.

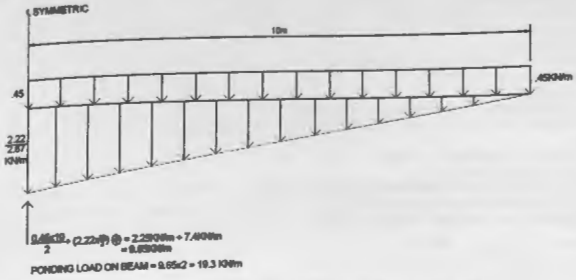


Figure 2.11. Resulting loads on sample roof due to ponding

Load on "Beam A" due to ponding = 19.3kN/m

Load on "Beam A" due to snow = 16.4kN/m

In this case ponding load is about 18% higher on the beam than the snow load. If the roof slopes were steeper (bringing more of the one-day rain to the centre of the roof), then the ponding effect would have been greater. When designing the roof deck in the valley area, the ponding load of 2.67kPa should be used rather than the snow load of 1.64kPa (an increase of over 60%).

Sample Output from a Snow Drift Calculation Program

```
Snow drifting calculation according to NBCC
Ground Snow Properties: Ss = 1.8 Sr = 0.2
Dimensions in meters of upper roof: Length = 7.0m Width = 5.0m
Dimensions in meters of lower roof: Length = 40.0m Width = 40.0m
Height in meters of roof step = 4.0m Upper Roof Parapet Height = 0.0m
Snow density to be used in calculations = 3.0kN/cubic meter
Building is Normal Importance: Importance factor for Snow = 1.00
Cw = 1.0 Cs (Upper Roof) = 1.0 Cs (Lower Roof) = 1.0

Basic roof snow according to NBCC
Compute characteristic length and basic snow of upper roof
Lc = 2*W-(W*W/L)
Lc = 2 *5.0 – (5.0*5.0 / 7.0) = 6.4 Characteristic Length Upper Roof = 6.4m
Compute Cb for upper roof using Cw = 1 formula Cb = 1-((30/Lc)^2)
Cb = 1 -((30 /6.4) ^2) = -20.8 Minimum value of 0.8 shall be used for Cb of upper roof
Snow load on upper roof = Is*(Ss*Cb*Cw*Ca + Sr)
Snow load on upper roof = 1.00 * (1.8 * 0.8 * 1.0 * 1.0 + 1.0 + 0.2) = 1.64kPa

Compute characteristic length and basic snow of lower roof
Lc = 2*W-(W*W/L)
Lc = 2 *40.0 – (40.0*40.0 / 40.0) = 40.0 Characteristic Length Lower Roof = 40.0m
Compute Cb for lower roof using Cw = 1 formula Cb = 1-((30/Lc)^2)
Cb = 1 -((30 /40.0) ^2) = 0.44Minimum value of 0.8 shall be used for Cb
```

```
Cb for Lower roof = 0.80
Snow load on Lower roof = Is*(Ss*Cb*Cw*Cs*Ca + Sr)
Snow load on Lower roof = 1.00 * (1.8 * 0.80 * 1.0 * 1.0 * 1.0 + 0.2) = 1.64kPa

Calculation of snow drift factor F (using NBCC provisions)
F = 0.35*Sqrt((Gamma*Lc/Ss - 6(Gamma*hp/Ss)^2))+Cb
F = 0.35 * Sqrt(10.71 - 6 * 0.00) + 0.80 = 1.946
This value is less than 2.0 Use minimum value of 2.00 for F
Ca(0) is lesser of:
Ca(0) = Gamma*h / (Cb*Ss) = 3.0 * 4.0 / (0.80*1.8) = 8.33
Ca(0) = F / Cb = 2.00 / 0.80 = 2.50
Governing Ca(0) = 2.50
Snow load on lower roof adjacent to step = Is*(Ss*Cb*Cw*Cs*Ca(0) + Sr)
Snow load on lower roof adjacent to step = 1.00 * (1.8 * 0.80 * 1.0 * 1.0 * 2.5 + 0.2) = 3.80kPa

Check Small step cut-off equation from NBCC
Ca(0) = Gamma*h / (Cb*Ss) = 8.33
This value exceeds Ca(0) calculated above and small step does not govern
Drift Distance from step = Xd = 5(Ss/Gamma)*(F-Cb)
Xd = 5(Ss/Gamma)*(F-Cb) = 5*(1.8 / 3.0) * (2.000 - 0.80) = 3.6

SUMMARY OF RESULTS:
Snow load on upper roof = Is*(Ss*Cb*Cw*Cs*Ca + Sr) = 1.64kPa
Snow load on lower roof away from step = Is*(Ss*Cb*Cw*Cs*Ca + Sr) = 1.64kPa
Snow loading on lower roof adjacent to building step = 3.8kPa
Distance on lower roof from step to uniform snow Xd = 3.6m
```

Notes: This output shows several desirable features:

1) Input data is given back in the output
2) Output contains a title
3) Output shows calculation date
4) The formulas used in the calculation form part of the output
5) The source of the formulas used is provided (Code references)
6) The numbers used in the formula are shown
7) A summary of results is provided

CHAPTER 3
BEAMS

The most common element in any building is the beam. Understanding the principles of steel beam design is a fundamental skill for any successful steel building designer. Not only is it important to design beams to use the steel efficiently, but the structural engineer must also design them in a way that uses our time most efficiently. Fortunately, the two aspects of designing efficiently are related.

3.1 PRINCIPLES AND EFFICIENCY OF BEAM DESIGN

Almost all floor beams for steel buildings will be designed using wideflange beams, as this is the most efficient use of the steel material. The majority of the weight of the beam is at the extremities where it is most effective in resisting the applied moments and giving the beam its greatest stiffness.

Wideflange beams are often designed using steel with a nominal yield stress of 345MPa rather than the 350MPa that we would initially expect. Using 345MPa reflects that most of the steel sections we use are produced in or for the U.S. market using American standards based on 50ksi material.

The yield stress used in beam selection tables (blue pages) in the *CISC Handbook* has changed twice in the past 20 years. In 1996, the standard beams went from 300MPa to 350MPa, and this was followed in 2000 by the tables adopting 50ksi (345MPa) when computing the resistance of wideflange sections. This change is illustrated in Table 3.1.

Table 3.1. Yield stress comparison over various editions of the *CISC Handbook*

CISC, 6th edition (1995)	M_r = 1450kNm (F_y = 300MPa)
CISC, 7th edition (1997)	M_r = 1690kNm (F_y = 350MPa)
CISC, 7th edition, 2nd printing (2000)	M_r = 1660kNm (F_y = 345MPa)
CISC, 9th edition (2006)	M_r = 1660kNm (F_y = 345MPa)
CISC, 10th edition (2010)	M_r = 1660kNm (F_y = 345MPa)

There is little difference between the Fy = 350MPa and Fy = 345MPa values, and designs using either would usually be acceptable. If a beam factored moment is just equal to the factored resistance using Fy = 350MPa, then consider going to a larger beam, as steel purchased by the fabricator may have been produced for the American market with a certified yield of 345MPa.

Most beams that are not part of a frame are sized as simple span beams, with the beam length being taken equal to the span between the centrelines of supporting elements, although the actual beam may be shorter. For normal rectangular framing using simple span members, the loading is computed using the assumption that all beams and deck segments are simply supported and have a constant tributary width throughout the span as shown in Figure 3.1.

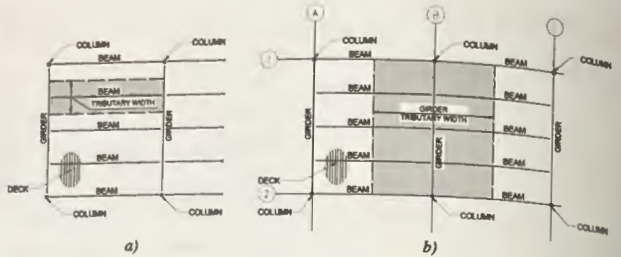


Figure 3.1. Typical tributary areas: a) beam b) girder

Beams should be selected by first using the beam resistance tables (blue pages) of the *CISC Handbook* (as shown in Table 3.2) and finding the minimum weight beam that satisfies the factored moment on the beam. While it is possible to compute beam capacities from requirements of the Standard and first principles, this is seldom done in practice.

Where possible, select the beam at the top of each group, as these beams will generally have the least weight for the desired capacity. For example, a laterally braced beam is desired with a moment capacity of 145kNm. Selecting a W360x33 with a moment capacity, M_r, of 168kNm is more efficient than a W250x39 with an M_r of 159kNm as it saves 6kg/m. Selecting the beam from the top of the group (in this case W360x33) will usually produce a more efficient design.

Table 3.2. Portion of beam resistance table

Designation	Vr	Ix	b	Lu	Mr
	kN	$10^6 mm^4$	mm	mm	<Lu
W360x33	396	82.6	127	1600	168
W250x39	354	60.1	147	2110	159
W310x33	423	65.0	102	1320	149
W200x46	300	45.2	203	3370	138
W200x42	302	40.6	166	2610	137

In some cases it will be necessary to adjust the beam capacity to match the restraint conditions. For most floor beams, the deck diaphragm is attached to the compression flange and we use M_r, the factored moment resistance for a fully supported beam. If the compression flange is not fully supported, then we need to use the reduced M_r' based on the unbraced length. The compression flange will not be fully supported in the following cases:

- Floor beams that support joists use L_u = joist spacing. Similar situations may exist where an extra layer of sloping beams is added over the main roof beams to provide support to a sloping roof deck while separating the roof deck diaphragm from the compression flange of the main roof beam.
- Cantilever beams where decking is attached to the top flange. (This will be covered in more detail later in the chapter.)
- Beam depth may be governed by architectural constraints and to minimize structural depth. This requirement may result in beams of heavier weight and lower depth being used. For example, if there is a maximum of 250mm available for beam depth, then W250 beams would be used even if W310 beams with less weight were structurally acceptable.

- It is preferable and more cost effective to have several similar sized beams on a job rather than choosing each beam to be minimum weight. If the design has only a few W310x24, but several W310x28 beams, then convert the W310x24 beams to W310x28.
- Flanges of beams that are to be located in walls should in general be a minimum of 25mm less than the stud size to be used for the wall.

Beams are to satisfy the deflection criteria given in Annex D of the Standard. Deflections are serviceability limit states and are therefore computed using unfactored loads and generally examine only individual load cases, not load combinations. For non-industrial buildings, floor and roof beams have a maximum deflection under live load = Span / 360. (Supposedly it is possible to go to L/300 for construction and finishes not susceptible to cracking, but it is suggested that this may be too flexible except in the case of some roof beams.)

Some engineers designing for snow conditions in areas of metro Vancouver, where it is recognized that the majority of the snow is wet and resistant to drifting, will concentrate on designing beams subject to snow drifting on the basis of strength. When doing serviceability checks, they will make sure that the deflection requirements for uniform snow have been addressed. Often, however, they are less concerned if the deflection of the beam under mandated drifting does not fully satisfy the deflection recommendations of Annex D.

Where large areas of floor are involved, or where the floor use is particularly sensitive to vibration, a vibration analysis should be performed.

Generally for vibration and serviceability the beam will follow minimum span-to-depth ratios and these can be used as a check on the design:

Roof Beams: Span / 25
Floor Beams: Span / 20

This is a rule of thumb; it is not a hard and fast rule. There are some conditions, such as very lightly or heavily loaded beams, where this rule of thumb does not provide accurate guidance. In these cases, the results of accurate analysis should not be rejected based only on span-to-depth ratios. Girders supporting beams should be of a depth equal to or greater the depth of the supported beam. For example, it is undesirable to have W250 girder supporting W360 beams.

EXAMPLE—DESIGN A BEAM IN A HIGH SCHOOL USING HANDBOOK DESIGN AIDS

Using data in Table 3.3 and Figure 3.2, design Beam A for strength and deflection.

Table 3.3. High school classroom loading

High school classroom loading - 65mm concrete on 38 deck			
Load Source	Dead (kPa)	Live (kPa)	Factored (kPa)
Assembly loading classrooms with or without fixed seats		2.4	3.60
Partition load / finishes / suspended mech	1		1.25
65 topping on 38 deck	1.89		2.36
Total load for Deck Design	2.89	2.4	7.21
Beams	0.15		0.19
Total load for Beam Design	3.04	2.4	7.40
Girders in either option (estimate)	0.15		0.19
Total load for Girder Design	3.19	2.4	7.59

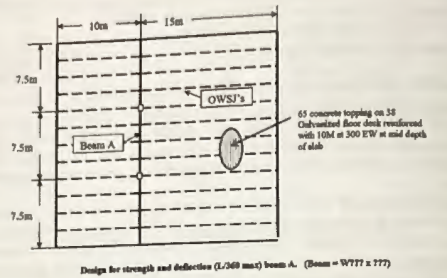


Figure 3.2. Design for strength and deflection (L/360 max) beam A

This should be a quick and easy calculation:

Compute the tributary width for the beam.

Trib. Width = (10m + 15m)/2

Trib. Width = 12.5m

Using the factored area load for girders from the load-sheet, compute the factored moment.

$$M_f = \frac{WL^2}{8} = \frac{7.59kPa * 12.5m * (7.5m)^2}{8} = 667kNm$$

Go to the beam selection tables (blue pages) in the *CISC Handbook* (as shown in Table 3.4) and find the most efficient beam that satisfies the required factored moment of 667kNm.

Table 3.4. Choosing a beam that satisfies the minimum moment requirement

BEAM SELECTION TABLE WWF and W Shapes				CSA G40.21 350W ASTM A992, A572 Grade 50	
Designation	V_r kN	I_x $10^6 mm^4$	b mm	L_u mm	M_r $\leq L_u$
W360x122	967	365	257	4040	705
W610x82	1170	560	178	2110	**683**
W460x97	1090	445	193	2650	677
W310x129	854	308	308	5080	671
W410x100	850	398	260	3730	661
W530x85	1130	485	166	2110	652
W530x82	1030	477	209	2660	640
W360x110	841	331	256	3940	640
W610x84	944	603	226	3000	627
W460x89	996	410	192	2620	624
W310x118	766	275	307	4920	605
W360x101	768	301	255	3860	584

The lowest beam in the table that satisfies the moment criteria is a W310x129, which has a moment resistance of 671kNm. Rather than choose that beam we go to the top of the grouping where a W610x82 has a moment resistance of 683kNm, but is substantially lighter. Select W610x82 as our trial beam size.

Using the trial beam size, compute the deflections under live load. If the deflection criterion (usually L/360) is satisfied then select that beam and go on to designing the next beam. If the deflection criterion is not satisfied then chose a beam at the top of the next grouping in the beam selection tables (blue pages) of the *CISC Handbook* and try it for deflection.

$$\Delta = \frac{5 * W_l * L^4}{384 * E * I}$$

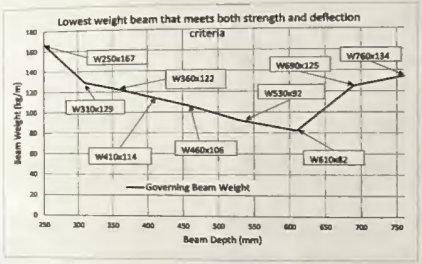
$$\Delta = \frac{5 * (2.4kPa * 12.5m) 7.5^4 * 1000}{384 * 200x10^6 * 560x10^{-6}}$$

$$\Delta = 11.0mm = L/680$$

Deflection and moment criteria are satisfied. Use W610x82.

This problem was given to 44 mostly practising engineers to do under the time constrained conditions of an exam; during actual design we are also subject to time constraints. Of the 44 engineers, 38 achieved the answer of W610x82. The remaining six had answers ranging from W610x91 (beam weight 11% higher than it needs to be) to W610x147 (beam weight 80% higher than it needs to be.

We can plot the most efficient beam size in the Figure 3.3.



Beam Depth (mm)	Minimum Weight Meeting Strength Criteria	Minimum Weight Meeting L/360 Deflection Criteria	Minimum Weight Meeting Both Criteria	Beam
250	149	167	167	W250x167
310	129	129	129	W310x129
360	122	122	122	W360x122
410	114	85	114	W410x114
460	106	74	106	W460x106
530	92	72	92	W530x92
610	82	82	82	W610x82
690	125	125	125	W690x125
760	134	134	134	W760x134

Figure 3.3. Beam economy for one particular beam

The six engineers who did not get W610x82 as their selected beam achieved beams that met the strength and deflection criteria. In an academic sense they had designed the beam correctly, but in a design office world their answers were "disappointing" since they had used more material than was necessary, which is neither acceptable from a "green" or a business competitive position. The typical oversights that result in heavier-than-necessary beams are:

- Picking a beam at random or possibly solely from span-to-depth ratios and checking strength and serviceability criteria. Designing this way takes longer, because several beams will need to be checked if a heavy section is not selected early in the process. The beam used as a trial section for the deflection check should be the most efficient beam that meets the moment criteria.
- Doing the deflection check using combined dead and live load. While this is often done in concrete and wood design as this best represents creep deflection, it is not the practice in steel beam design. For floor beams we compare the deflection computed using only unfactored live load and comparing that to the deflection criteria.

EXAMPLE—DESIGN A BEAM IN A HOTEL USING HANDBOOK DESIGN AIDS

Using data in Table 3.5, design a steel beam located in a hotel lobby floor. The beam span is 8.5m and the tributary width is 2.5m. The deck is 76mm deep with 65mm concrete topping. There are no architectural restrictions as to beam depth. The maximum permitted deflection is L/360.

Table 3.5. Assembly live load

LOAD SOURCE	DEAD (kPa)	LIVE (kPa)	FACTORED (kPa)
Assembly loading (Code Table 4.1.5.3)		4.8	7.2
Partition load / finishes / suspended mech. & elect.	1.0		1.3
65 topping on 76 deck	2.1		2.6
Total load for Deck Design	3.1	4.8	11.1
W410 beam at 2500 o/c	0.2		0.3
Total load for Beam Design	3.3	4.8	11.3
W610 beam at 8500 o/c	0.1		0.1
Total load for Girder Design	3.4	4.8	11.5

Factored Beam Load = Factored area loading * Tributary width
= 11.3kN/m² * 2.5m = 28.25kN/m

$$\text{Factored Beam Moment} = M_f = \frac{WL^2}{8} = \frac{28.25kN/m*(8.5m)^2}{8} = 255kNm$$

Using the beam selection tables (blue pages) from the *CISC Handbook* (as shown in Table 3.6), we can select the most efficient beam that will resist the moment.

Table 3.6. Beam selection tables from the *CISC Handbook*

Designation	Vr	I_x	b	L_u	M_r
	kN	10^6mm^4	mm	mm	$\leq L_u$
W360x51	**524**	**141**	**171**	**2320**	**278**
W410x46	**578**	**156**	**140**	**1790**	**275**
W310x52	495	119	167	2370	261
W200x71	452	76.6	206	3730	249
W360x45	**498**	**122**	**171**	**2260**	**242**

M_r = 255kNm

The lowest beam in the table that has a factored moment resistance of 255kNm is a W310x52 with a moment resistance of 261kNm. Rather than trying the W310x52 we will try the beam at the top of the group (W410x46) that is highlighted in bold, as this will give the most efficient beam size from a weight standpoint.

For a W410 x 46, M_r = 275kNm when the top flange is fully supported or supported at a spacing of less than L_u. We will look at unsupported beams later in the chapter.

We will now check that the beam meets serviceability criteria by checking that its deflection under live load is less than the span of the beam divided by 360.

$$\Delta = \frac{5*W_l*L^4}{384*E*I} = \frac{5*(4.8kPa*2.5m)*8.5^4*1000}{384*200*156} = 26.1mm = L/325$$

A deflection under live load of L/325 is too flexible and we will have to increase the size of the beam such that the inertia is greater and deflections are lower. Deflections are a linear function of the inertia and we can take our result from the calculation above and multiply the moment of inertia for the beam tried by the ratio of 360 divided by the value found to get the moment of inertia necessary to satisfy live load criteria:

Want I_x = (360/325)*156 = 172

Could use W410x54 (I_x = 186) or W460x52 (I_x = 212)

As there is no architectural depth limitation, select the lower weight beam and use W460x52.

Now the beam is checked for shear:

V_f = Factored Area load * Tributary Width * (Span Length / 2)

V_f = 11.3kPa * 2.5m * (8.5m / 2) = 120kN (V_r = 680kN) OK

Note: V_f need seldom be checked for most uniformly loaded wide-flange sections.

Final Check: Span / Depth = 8500mm / 460mm = 18.5 (OK – should be about 20).

RESULT: Use W460x52.

EXAMPLE—DESIGN A GIRDER USING HANDBOOK DESIGN AIDS

Using data in Table 3.7, design a girder that supports OWSJs for a school classroom that has 76mm deep steel deck with 65mm of concrete topping. The girder spans 10m and supports joists at 2.5m o/c. The top flange of the beam is restrained only at joist locations. The joist span is 9m on one side and 8m on the other side of the girder. Architectural restrictions give a maximum beam depth as 550mm. The maximum permitted deflection is L/360.

Table 3.7. Classroom live load

LOAD SOURCE	DEAD (kPa)	LIVE (kPa)	FACTORED (kPa)
Classroom Live load (Code Table 4.1.5.3)		2.4	3.6
Partition load / finishes / suspended mech. & elect.	1.0		1.3
65 topping on 76 deck	2.1		2.6
Total load for Deck Design	3.1	2.4	7.5
Joists at 2500 o/c	0.1		0.1
Total load for Joist Design	3.2	2.4	7.6
W530 girder at 8500 o/c	0.2		0.3
Total load for Girder Design	3.4	2.4	7.9

Tributary Width = (Span Left + Span Right) / 2 = (8.0m + 9.0m) / 2 = 8.5m

Factored Beam Load = Factored area loading * Tributary width
= 7.9kN/m² * 8.5m = 67.1kN/m

$$\text{Factored Beam Moment} = \frac{WL^2}{8} = \frac{67.1kN/m*(10m)^2}{8} = 839kNm$$

Try a W530x109, with M_r at L_u = 2.5m = 879kNm
(Note: W610x101, with M_r = 900kNm at L_u = 2.5m, is more efficient but violates architectural depth criteria.)

Compute deflection under service live load.

$$\Delta_{LL} = \frac{5*W_l*L^4}{384*E*I} = \frac{5*(2.4KPa*8.5m)*10.0^4*1000}{384*200*667} = 19.9mm = L/502$$

Check Shear: V_f = Factored Area load * Tributary Width * (Span Length / 2)

V_f = 7.9kPa * 8.5m * (10.0m / 2) = 336kN (V_r = 1300kN)

Final Check: Span / Depth = 10,000mm / 530mm = 18.9 (OK – should be about 20).

RESULT: Use W530x109.

Note: It could be argued that the loading should have been treated as point loads from the joists rather than a uniform load; however, using a uniform load is computationally easier, especially when computing deflections. The accuracy of this simplification improves as the number of beams or joists picked up by the girder increases – three or more point loads on a girder will produce essentially identical results with either a point loaded or uniform loaded model.

The next few sections discuss the design of beams in more detail and from first principles. Beams falls into two categories from a design standpoint: those with their compression flanges supported laterally and those with unsupported compression flanges. In both cases, we start by determining the "Class" of the wide-flange beam that we will be using for typical beams, as the class of beam has an impact on the resistance of both supported and unsupported beams.

3.2 SECTION CLASSES FOR WIDE-FLANGE MEMBERS

The definitions of section classes are given in S16 Clause 11.1.1 and shown in Table 3.8.

Table 3.8. Objectives for the various beam classes

CLASS	OBJECTIVE
C1	Permit attainment of the plastic moment and subsequent redistribution of the bending moment. (Sometimes referred to as "plastic design sections.")
C2	Permit attainment of the plastic moment but need not allow for subsequent moment redistribution. (Sometimes referred to as "compact sections.")
C3	Permit attainment of the yield moment. (Sometimes referred to as "non-compact sections.")
C4	Generally have elastic local buckling of elements in compression as the limit state of structural resistance. (Sometimes referred to as "slender sections.")

Many clauses in S16 rely on the class of member involved and will give the designer advantages when designing with members that exhibit good behaviour. Good behaviour is defined as being able to develop

plastic moments without local buckling. Conversely, the design rules associated with Class 4 sections can be lengthy to apply.

For a wide-flange beam, the rules are set out in the Commentary to the Standard as shown in Figure 3.4.

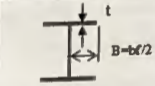
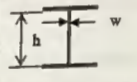
Detail	Class 1	Class 2	Class 3
	$\frac{b}{t} \le \frac{145}{\sqrt{Fy}}$	$\frac{b}{t} \le \frac{170}{\sqrt{Fy}}$	$\frac{b}{t} \le \frac{200}{\sqrt{Fy}}$
	$\frac{h}{w} \le \frac{1100}{\sqrt{Fy}}$	$\frac{h}{w} \le \frac{1700}{\sqrt{Fy}}$	$\frac{h}{w} \le \frac{1900}{\sqrt{Fy}}$

Figure 3.4. Design rules for a wide-flange beam

Note that Clause 11.3.1(c) of S16 states that for flanges of beams and tees, the width, b_{el}, is taken as one-half of the full nominal dimension. This means we take the flange width / 2, not the distance the flange projects beyond the web.

EXAMPLE—DETERMINE THE BENDING CLASS

Determine the class in bending of a W410x39 made of steel with F_y = 345MPa, as shown in Figure 3.5.

Solution 1:

Flange width, b_f = 140; Flange thickness, t = 8.8

$b_{el} = b_f / 2 = (140) / 2 = 70$

$b_{el} / t = 70 / 8.8 = 7.95$

For Class 1: $b_{el} / t = 145 / \sqrt{F_y} = 145 / \sqrt{345} = 7.81$ (Does not satisfy)

For Class 2, $b_{el} / t = 170 / \sqrt{F_y} = 170 / \sqrt{345} = 9.15$ (OK)

Check web requirements

Web thickness = w = 6.4mm Clear distance = d–2t = h = 381mm

h / w = 381 / 6.4 = 59.5

For Class 1: $h / w = 1100 / \sqrt{F_y} = 1100 / \sqrt{345} = 59.2$

(close, but web does not satisfy Class 1)

For Class 2: $h / w = 1700 / \sqrt{F_y} = 1700 / \sqrt{345} = 91.5$ (OK)

In this case, the section class is governed by both the web and flange condition; the section is

Class 2. If the classes of the flanges and web are different, the overall section is classified by the more slender classification.

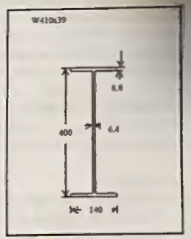


Figure 3.5. Flange width and thickness

Solution 2:
Consult Table 5-1 of the *CISC Handbook*. W410x39 for this grade of steel is Class 2 for bending.

3.3 BENDING CAPACITY OF LATERALLY SUPPORTED BEAMS

The bending capacity of laterally supported beams is covered in Clause 13.5 of S16. The design process can be expressed by the flowchart in Figure 3.6.

S_e is the effective section modulus for the beam, neglecting the portions of the beam flanges that are too far away from the web to be effective. If you need to follow this path, you will need to read S16 in much more detail. Ø is the resistance factor, which for this purpose is 0.9. We will cover Ø factors in more depth.

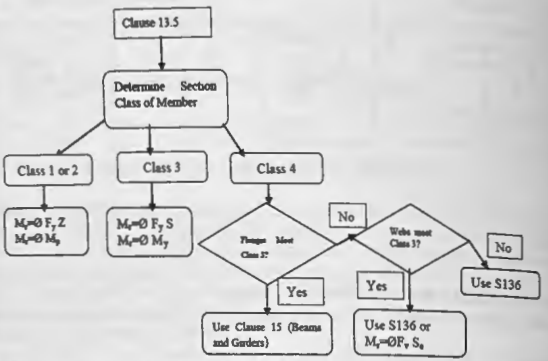


Figure 3.6. Flowchart to determine bending capacity of laterally supported beams

The flowchart shows the advantage of Class 1 or 2 over Class 3 or 4. Because Class 4 can lead to extensive calculations and reference to other standards, it is generally advantageous for the designer to stay away from Class 4 beams. These rules are accounted for in the beam resistance tables (blue pages) of the *CISC Handbook*, with M_r for various classes of beams being computed using the appropriate elastic or plastic section modulus. Two examples from the tables are shown here.

EXAMPLE—FLEXURAL DESIGN OF A CLASS 2 LATERALLY SUPPORTED BEAM

We have determined that a W410x39 section with $F_y = 345$MPa steel is a Class 2 section. Determine the moment capacity for this beam if the compression flange is laterally restrained.

Solution 1:
Determine from first principles.

As a Class 2 section, $M_r = \emptyset F_y Z$, which is equal to $M_r = \emptyset M_p$.

We get $Z = 730x10^3mm^3$ from the W-shapes table in Part 6 of the *CISC Handbook*.

$Z = 730x10^3mm^3 * 10^{-9}mm^3/m^3$

$Z = 730x10^{-6}m^3$

$M_r = 0.9 * 345{,}000kN/m^2 * 730x10^{-6}m^3$

$M_r = 227kNm$

Solution 2
Consult the moment capacity tables set out in the beam resistance tables (blue pages) of the *CISC Handbook* and find $M_r = 227$kNm.

EXAMPLE—FLEXURAL DESIGN OF A CLASS 3 LATERALLY SUPPORTED BEAM

Table 5-1 of the *CISC Handbook* shows us that a W250x18 is Class 3 if using 345MPa steel. Determine the moment capacity for this beam if it is laterally restrained.

Solution 1:
Determine from first principles.

As a Class 3 section, $M_r = \emptyset F_y S$, which is equal to $M_r = \emptyset M_y$.

We get $S = 179x10^3mm^3$ from the W-shapes table in Part 6 of the *CISC Handbook*.

$S = 179x10^3mm^3 * 10^{-9}mm^3/m^3$

$S = 179x10^{-6}m^3$

$M_r = 0.9 * 345{,}000kN/m^2 * 179x10^{-6}m^3$

$M_r = 55.6kNm$

Solution 2:
Obtain the capacity from the moment capacity tables (blue pages) of the *CISC Handbook* and find $M_r = 55.6$kNm. (Note that in these tables Class 3 sections are identified by dagger symbol [†].)

3.4 BENDING CAPACITY OF LATERALLY UNSUPPORTED BEAMS

Beams that have their compression flange unsupported are prone to lateral-torsional buckling, and the bending capacity of laterally unsupported beams reduces rapidly as the unsupported length of the beam increases. L_u is defined in S16 as the "longest unbraced length with which a beam will reach either $M_r = \emptyset M_p$ or $M_r = \emptyset M_y$, depending on the class of the cross-section." This implies that as long as the unbraced length $L<L_u$, there is no reduction in the factored moment resistance M_r. This is shown in Figure 3.7, which shows the moment resistance of an unsupported W410x39. The capacity drops rapidly when the unsupported length exceeds the L_u value for the beam.

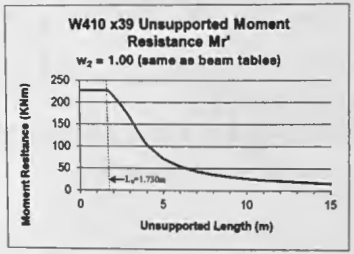


Figure 3.7. Moment resistance of an unsupported W410x39

The unsupported length of the beam must be identified correctly to get the correct bending moment. The following points should be observed:

- In a beam with reversing curvature, the points where the moment changes from positive to negative (points of contraflexure) are not support points unless the flange is physically restrained at that point either by a brace or attachment to a diaphragm.

- In cantilever beams, the unsupported length can be several times the length of the cantilever (we will cover this important topic later in the chapter).
- In simple span beams where there is no support to the compression flange (e.g., roof beams under wind uplift), the unsupported length of the beam will often be the entire length of the beam.

The bending capacity of laterally unsupported beams (M_r' in the beam selection tables of the *CISC Handbook*) is covered in Clause 13.6. The design process can be expressed for doubly-symmetric beams by the flowchart shown in Figure 3.8.

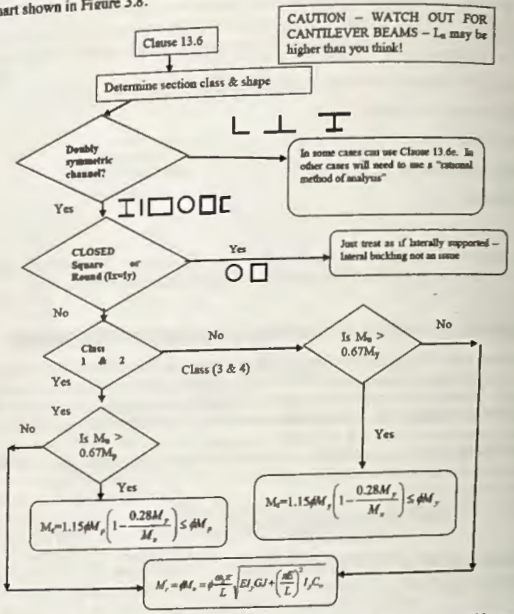


Figure 3.8. Flowchart to determine bending capacity of laterally unsupported beams

The value of ω_2 depends on the moment gradient and is defined as follows:

$$\omega_2 = \frac{4M_{max}}{\sqrt{M_{max}^2 + 4M_a^2 + 7M_b^2 + 4M_c^2}}$$

where: M_{max} = maximum factored bending moment magnitude in the unbraced segment
M_a = Factored bending moment at one-quarter point of the unbraced segment
M_b = Factored bending moment at midpoint of the of the unbraced segment
M_c = Factored bending moment at three-quarter point in the unbraced segment

For a uniformly loaded, simply supported beam, the value of ω_2 can be shown to be 1.13, as shown in the following example.

EXAMPLE—DETERMINATION OF ω_2

Let's examine this on a simply supported beam with uniform load and unbraced between the beam ends, as shown in Figure 3.9.

$$M_{max} = M_b = \frac{wl^2}{8} = 0.125wl^2$$

$M_a = M_c = 0.09375wl^2$ (from free body of end; see Figure 3.9 at right)

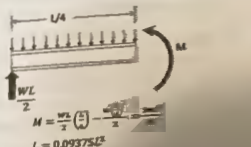


Figure 3.9 Simply supported beam

$$\omega_2 = \frac{4M_{max}}{\sqrt{M_{max}^2 + 4M_a^2 + 7M_b^2 + 4M_c^2}}$$

$$= \frac{4 * 0.125}{\sqrt{0.125^2 + 4 * 0.09375^2 + 7 * 0.125^2 + 4 * 0.09375^2}} * \left(\frac{wl^2}{\sqrt{(wl^2)^2}}\right)$$

$$\omega_2 = \frac{0.5}{\sqrt{8 * 0.125^2 + 8 * 0.09375^2}} = \frac{0.5}{\sqrt{0.125 + 0.0703}} = \frac{0.5}{0.442} = 1.13$$

Note that the value of ω_2 depends only on the distribution of the moment (shape of the bending moment diagram), not on its magnitude.

The beam selection tables (blue pages) in the *CISC Handbook* contain values of M_r' for various unsupported lengths; however, these tables use ω_2 equal to 1.0 and therefore a modification is required when using the tables. For the most part this modification will be multiplying the M_r' value from the table by ω_2, while being careful not to exceed the moment capacity of the beam in the fully supported condition. The equation for ω_2 assumes that the load is applied at the shear centre (conservative for loads applied below the shear centre). If the load is applied above the shear centre, then a "rational analysis" must be performed, though in the case of loads applied at the top (compression flange) level, S16 allows us to use an effective length of 1.2L for pin-ended beams and 1.4L for other cases on in-plane flexural restraint (with $\omega_2 = 1.0$).

The use of a ω_2 value of 1.13 produces a slight increase in capacity, provided that the unsupported length of the beam is sufficiently high that we are away from the region where the plastic or yield moment governs. This is shown in Figure 3.10, which shows the results for a W410x46 with two ω_2 values.

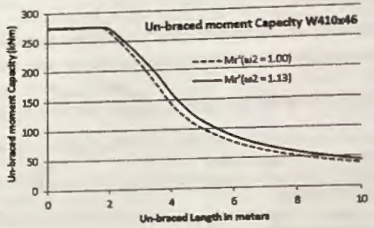


Figure 3.10. Moment capacities for an unbraced W410x46 with two ω_2 values

EXAMPLE—DESIGN OF LATERALLY UNSUPPORTED BEAM

Determine M_r' of a uniformly loaded W410x46 using 345MPa steel and with the compression flange unsupported over a length of 8m. The load is applied at the shear centre or on the tension side of the shear centre of the beam. Use ω_2 of 1.13.

Solution 1:

Go to the beam selection tables (blue pages) of the *CISC Handbook* and determine $M_r' = 51.8$kNm. We then multiply this by 1.13 to get 58.5kNm. Confirm that this does not exceed $M_r = fM_p = 275$kNm.

Solution 2:

Apply the S16 provisions:

Table 5-1 gives the class of section in bending. W410x46 in 345MPa steel is a Class 1 beam.

As 8m is a long unsupported length, we will assume (and check later) that $M_u < 0.67M_p$.

$$M_r = \phi \frac{\omega_2 \pi}{L} \sqrt{EI_y GJ + \left(\frac{\pi E}{L}\right)^2 I_y C_w}$$

As the beam is uniformly loaded, ω_2 is 1.13.

Substitute values for the W410x46 beam using values from the W-shapes table:

$I_y = 5.14x10^6 mm^4$

$J = 192x10^3 mm^4$

$C_w = 197x10^9 mm^6$

$E = 200kN/mm^2$, $G = 77kN/mm^2$

$\omega_2 = 1.13$ for uniformly loaded beams

Place these values in the equation.

$$M_r = 0.9 * \frac{1.13 * \pi}{8000} * \sqrt{200 * 5.14x10^6 * 77 * 192x10^3 + \left(\frac{\pi * 200}{8000}\right)^2 * 5.14x10^6 * 197x10^9}$$

$$M_r = 3.99x10^{-4} * \sqrt{1.52x10^{16} + 0.625x10^{16}} = 3.99x10^{-4} * 1.46x10^8$$

$M_r = 58.3x10^3 kNmm = 58.3kNm$

Check if this is less than $0.67*M_p$

$0.67M_p = 0.67*(345{,}000kN/m^2)*(885x10^3 mm^3)(x10^{-9}) = 205kNm$

OK – correct formula used. Note that the calculation agrees with the value of 51.8kNm found in the tables.

Note the long-form method of calculating the unbraced moment capacity is only done in the following circumstances:

- We are working with a section that is not listed in the tables. Such a section may occur if a custom section is desired for architectural reasons.
- We are using a section such as a channel with greater depth than 250mm or a rectangular HSS that is not covered in the tables.
- We are writing a computer program or spreadsheet to solve this type of problem.
- We are trying to illustrate how the S16 provisions work for a case where the answer is known.

Figure 3.11 shows the unbraced moment capacity of a W410x46 beam at various unsupported lengths. The moment resistance curve is in three parts – at short unsupported lengths the material capacity governs

(solid line at $\emptyset F_y$); at large unsupported lengths the elastic buckling capacity governs (solid curved line); and at intermediate values there is a transition curve (dashed line) characterized by inelastic lateral-torsional buckling.

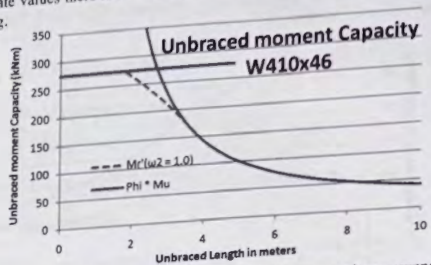


Figure 3.11. Unbraced moment capacity of a W410x46 beam at various unsupported lengths

For beams that have their compression flange unsupported and that are subjected predominantly to end moments at the points of lateral restraint, a simpler formulation for ω_2 is permitted (although the previous equation is also acceptable). These members will have essentially linear moment diagrams with the shape of the diagram defined exclusively by the end moments and, if the end moments are opposite, then the section will have a point of inflection in the beam.

The equation for ω_2 for a beam with predominantly end moments is:

$$\omega_2 = 1.75 + 1.05\kappa + 0.3\kappa^2 \le 2.5$$

Where $\kappa = \frac{M_{smaller}}{M_{larger}}$ (Positive for double curvature – Negative for single curvature.

Double curvature has an inflection point within the span.)

If $M_{small} = 0.5*M_{large}$ then $\kappa = \frac{M_{smaller}}{M_{larger}} = 0.5$ (Positive, as point of inflection in span.)

$$\omega_2 = 1.75 + 1.05(0.5) + 0.3(0.5)^2 = 2.3 \text{ (OK } < 2.5)$$

The relationship between κ and ω_2 is shown in Figure 3.12. As the largest magnitude of M_{small} can only be equal to M_{large} ω_2 therefore has the range of values of –1.0 (constant moment along member) to +1.0 (equal and opposite moments with a point of inflection in the centre of the unbraced length).

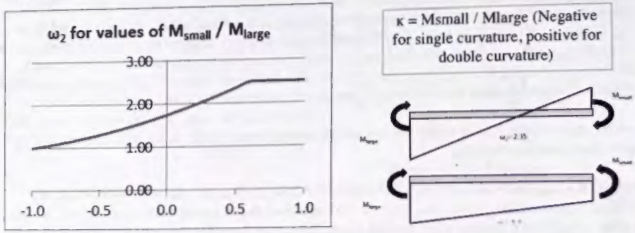


Figure 3.12. Relationship between κ and ω_2

If we compare the moment capacity for an unbraced beam with single curvature (k = -0.5) to the moment capacity with double curvature (k = 0.5), we can see that the unbraced moment capacity is improved dramatically by having an inflection point in the span, as shown in Figure 3.13. The benefit of double curvature diminishes as the unbraced length decreases.

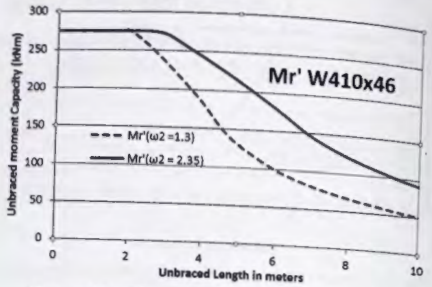


Figure 3.13. Unbraced moment capacity for W410x46

3.5 LATERALLY UNSUPPORTED PLATES UNDER UNIFORM LOAD

One case that is worth examining is the case of the bending capacity of a plate that is laterally unsupported. This can be useful when examining "architectural" steel items where unsupported fin plates may be used. It may also be reasonable to think of a tee section as a single vertical plate and to design the tee using the same stress that is found acceptable for a fin plate of thickness equal to the web acting without the flange.

Like all sections that pass through a single point (including angles and tees), the single plate has a C_w value of 0. Having $C_w = 0$ considerably simplifies the equations.

$$M_r = \phi \frac{\omega_2 \pi}{L} \sqrt{EIyGJ + \left(\frac{\pi E}{L}\right)^2 IyCw} = \text{(in the case of } C_w = 0)$$

$$M_r = \phi \frac{\omega_2 \pi}{L} \sqrt{EIyGJ}$$

J for a plate $= \frac{bt^3}{3}$, where t is the smaller dimension.

$$I_y = \frac{bt^3}{12}$$

Applying these we get:

$$M_r = \emptyset M_u = \phi \frac{\omega_2 \pi}{L} \sqrt{EIyGJ} = \phi \frac{\omega_2 \pi}{L} \sqrt{E \frac{bt^3}{12} G \frac{bt^3}{3}}$$

$$M_r = \phi \frac{\omega_2 \pi}{L} \sqrt{E \frac{b^2 t^6}{36} G} = \phi \frac{\omega_2 \pi * b * t^3}{L} \sqrt{\frac{EG}{36}}$$

The value must be checked to verify that we are not greater than $0.67M_y$, as the equation above is only valid for moments less than this.

EXAMPLE—DESIGN A LATERALLY UNSUPPORTED PLATE (LOADED IN-PLANE)

Determine the moment resistance of a 12.7mm x 104mm plate that is laterally unsupported over 3500mm. The plate is uniformly loaded in the vertical plane. Determine the maximum stress in the plate when it reaches this moment resistance. The plate is from G40.21 300W steel.

$$M_r = \phi \frac{\omega_2 \pi * b * t^3}{L} \sqrt{\frac{EG}{36}}$$

$$= \phi \frac{1.13\pi * 104mm * (12.7mm)^3}{3500mm} \sqrt{\frac{200kN/mm^2 * 77kN/mm^2}{36}}$$

$M_r = \emptyset M_u = \phi$ (4469kNmm) = ϕ (4.469kNm) = 4.02kNm

Determine if this is >0.67My

$$M_y = F_y * S = F_y * \frac{b^2 t}{6} = 0.300 \text{ kN/mm2} * \frac{104^2 mm * 12.7mm}{6} = 6868\text{kNmm}$$

0.67 M_y = 0.67 * 6.868kNm = 4.602kNm OK

The stress at maximum moment =

$$\frac{M}{S} = \frac{4.02kNm}{\frac{0.104^2 m * (0.0127m)}{6}} = 175500kN/m^2 = 175MPa$$

3.6 BENDING CAPACITY OF LATERALLY UNSUPPORTED BEAMS WITH SINGLY-SYMMETRIC CROSS-SECTIONS

While steel beams with doubly-symmetric cross-sections are by far the most common in typical steel-frame construction, beams with singly-symmetric (monosymmetric) cross-sections arise regularly. S16 treats the two cases similarly, but the lack of cross-section symmetry creates some complexity. The first thing to realize is that the treatment depends on which axis is the axis of symmetry with respect to the bending axis. Singly-symmetric beams that bend about the axis of symmetry (e.g., a channel bent about the strong axis or a wideflange shape with unequal flange widths bent about the weak axis), are dealt with in the same way as doubly-symmetric shapes. Singly-symmetric beams that bend in the plane of the axis of symmetry must account for the difference in lateral-torsional buckling behaviour due to the asymmetry. The elastic LTB equation contains mostly the same parameters as the one for doubly-symmetric shapes, but introduces a cross-sectional property, β_x, which is sometimes referred to as the "coefficient of mono-symmetry." The elastic buckling equation is:

$$M_u = \frac{\omega_1 \pi^2 E I_y \beta_x}{2L^2} \left[1 \pm \sqrt{1 + \frac{4}{\beta_x^2} \left(\frac{GJL^2}{\pi^2 E I_y} + \frac{C_w}{I_y} \right)} \right]$$

The ± sign before the radical indicates that due to the asymmetry, there are two critical moments, one positive (↻) and one negative (↺). Thus the value of the critical moment depends on the direction in which the moment acts. Note that the value under the radical is always greater than 1.0, so the value in the square brackets will give a larger positive value than negative.

The value of β_x depends only on the cross-sectional proportions of the beam and the sign depends on the direction of the applied moment. In general, take the sign to be positive if the (laterally) stiffer flange is in compression and negative if it is in tension. For example, for a T-section (written for positive bending in the plane of symmetry and shown in Figure 3.14).

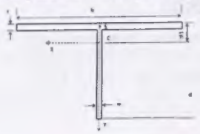


Figure 3.14. T-section

$$\beta_x = \frac{1}{I_x}\left\{\frac{w}{4}\left[(d-\bar{y}_j)^4 - \left(\bar{y}_T - \frac{t}{2}\right)^4\right] - bt\left(\bar{y}_T - \frac{t}{2}\right)\left[\frac{b^2}{12} + \left(\bar{y}_T - \frac{t}{2}\right)^2\right]\right\} - 2y_0$$

$$y_0 = -\left(\bar{y}_T - \frac{t}{2}\right) = \text{coordinate of shear centre (negative)}$$

S16 includes an approximate equation that may be used to determine β_x in lieu of a more accurate value, as follows:

$$\beta_x = 0.9(d-t)\left(\frac{2I_{yc}}{I_y} - 1\right)\left(1 - \left(\frac{I_y}{I_x}\right)^2\right)$$

which for a T-section collapses to (sign depending on whether the flange or stem is in compression)

$$\beta_x = \pm 0.9(d-t)\left(1 - \left(\frac{I_y}{I_x}\right)^2\right)$$

but the approximate equation is less accurate for T-sections than for beams with two flanges.

By accounting for the sign of β_x as described above, the moment capacity obtained is simply a magnitude, larger if the stiffer flange is in compression and smaller if it is in tension. Some designers find it more intuitive to consider β_x simply as a cross-sectional property (without sign), which will yield two moment capacity magnitudes with opposite signs, where the positive (larger) value is the relevant capacity when the stiffer flange is in compression as is usually the case. If it is not the case, the negative (smaller) value is used. It is helpful to note that for doubly-symmetric sections, $\beta_x = 0$ and the elastic LTB equation above collapses to the more familiar one presented in Section 3.4.

For beams with singly-symmetric cross-sections, ω_3 serves an analogous function to ω_2 for doubly-symmetric sections. For beams bent in single curvature, $\omega_3 = \omega_2$ for sections with two flanges, but is taken as 1.0 for T-sections. For all other cases, $\omega_3 = \omega_2(0.5 + 2(I_{yc}/I_y)^2)$, but not greater than 1.0 for T-sections, where I_{yc} is the moment of inertia of the compression flange about the y-axis.

In design, the boundary between elastic and inelastic LTB is delineated for beams with singly-symmetric cross-sections by the quantity $M_{yr} = 0.7S_xF_y$ (the length at which this capacity occurs is symbolized by L_{yr}), with S_x taken as the smaller of the two potential values. Therefore, for $M_u \le M_{yr}$:

$$M_r = \phi M_u$$

For $M_u > M_{yr}$:

$$M_r = \phi\left[M_p - (M_p - M_{yr})\left(\frac{L_p - L_u}{L_{yr} - L_u}\right)\right] \le \phi M_p$$

with M_p replaced by M_y for Class 3 sections and Class 1 and 2 T-sections where at any point within the unbraced segment the stem tip is in compression. The equation is a simple linear transition to account for partial yielding of the cross-section prior to buckling, including the effect of residual stresses. The quantity L_u, which is conveniently tabulated for standard shapes in the *CISC Handbook*, must be estimated and S16 provides an approximate equation to do so.

3.7 BRACING OF INDIVIDUAL MEMBERS

We have seen how the capacity of a beam in bending drops rapidly as the length between the points of lateral restraint of the compression flange increases. For most beams in buildings under downward loads, the top flange is supported by the roof or floor diaphragm. This is shown in Figure 3.15.



a) b)

Figure 3.15. a) A deck diaphragm that is assumed to provide continuous support to the top flange; each flute of the deck has been connected to the beam. b) A deck diaphragm that is fastened to the joists, which are separated from the top flange of the beam by the joist seat. The diaphragm is therefore only stabilizing the top flange of the beam at the joist seat locations (see circled sections)

The stability of individual members can be assessed and the bracing forces necessary to provide lateral restraint calculated in two ways, one easy and straightforward and the other more logical but more complex.

To determine what forces are needed to stabilize members so that they do not buckle, two methods of calculating bracing forces are now given in Clause 9 of S16. The first is given in Clause 9.2.5 as the simplified method. This requires that the brace be capable of resisting 2% of the compression force being braced.

The second method of computing bracing forces is the direct method, and is given in Clause 9.2.6.2 as follows:

$$P_b = \frac{\beta[\Delta_o + \Delta_b]C_f}{L}$$

where:

P_b = Force in brace

Δ_o = Initial misalignment, taken as the tolerance for sweep or camber from Clause 28.6; a table showing the sweep and camber information can be found in Part 6 of the *CISC Handbook*. The variation in straightness for W-shapes with flanges greater than 150mm is L / 1000.

Δ_b = Displacement of the bracing system assumed to be equal to Δ_o for first-order analysis.

β = a number dependent on the number of braces, as shown in Table 3.9.

Table 3.9. Number of brace points

NUMBER OF BRACE POINTS	β
1	2
2	3
3	3.41
4	3.63
Greater than 4	4

C_f = Compression force being braced. In a beam, it would typically be the force in the compression flange. In a column, it would be the axial force in the column.

L = Length between bracing points.

This seems overly complex for determining normal brace forces, and may in some cases be iterative.

EXAMPLE—DETERMINE DISCRETE BRACE FORCE

For a 10m long W530x66 beam made from 350W steel, if it is braced at third points (two braces to the top flange) find the corresponding force in the braces.

Solution:

Design the brace to resist the maximum compression force in the flange.

Flange force = 165 * 11.4 * 350 * 0.9 * 10^{-3} = 593kN

Initial sweep of beam = 0.001 * L = 0.001 * 10,000mm = 10mm

Braced at third points: $P_b = \frac{(\beta = 3.0)*(0.010m + 0.010m)*(Cf = 593kN)}{(L = 10m / 3 = 3.33m)} = 10.7kN$

10.7kN / 593kN = 1.8% of compression flange force. This is slightly less than the 2% force permitted by the S16 simplified method.

Most designers will elect to use the S16 simplified procedure (Clause 9.2.5) where it is only necessary to brace 2% of the compression force. For our 10m long W530x66, the bracing force would be 0.02*593kN = 11.9kN.

Clause 9.2.7 requires that beams stabilized by a slab or deck be stabilized by a force equal to 5% of the flange force distributed over the length of the beam. As the flange force is independent of beam span, the stabilization of short beams may be more difficult.

EXAMPLE—DETERMINE DISTRIBUTED BRACE FORCE

A 0.91mm steel roof deck is used to stabilize the top flange of a W410x67 steel beam made from 350W steel. Determine the force required to stabilize the beam and the approximate number of puddle welds required.

Solution:

The W410x67 beam has a flange force of:

(b = 0.179m) x (w = 0.0144m) x (F_y = 350,000kN/m²) x (ϕ = 0.9) = 812kN

When using deck the stabilizing force = 0.05 * 812kN = 40.6kN

With a factored resistance of 6.8 kN for each puddle weld in 0.91mm deck, this requires 40.6kN / 6.8kN = 5.9, say six puddle welds. If puddle welds are at 200mm on centre, this would be achieved by any beam 1.4m or longer.

Bracing will often provide restraint to the compression flange of more than one beam, truss, or joist. A brace to more than one beam may be able to take advantage of the low probability that the random out-of-alignments of the members will all be in the same direction. The suggested approach is to follow S16 Clause 9.2.8, which gives an initial out-of-straightness value for the group of members that can then be put into the direct method. There is not much guidance provided for the accumulation of forces in the brace when the simplified method of calculating brace forces is used. For joists, an examination of Clause 16.7 (Bridging) is required.

There will be some instances where a structural steel system has no diaphragm. One example would be a "floor" of steel beams supporting steel grating. The grating comes in panels and is connected to the floor beams often with clips or with tack welds. The system is not considered a diaphragm as there are no provisions for providing a shear transfer from one panel to the next and sections of grating panels may be removed in the future. To be considered a diaphragm it is necessary that the diaphragm material have a competent load path from the flange that is trying to buckle to a point of support such that the flange

cannot move relative to the supported ends of the beam. To reduce the unbraced length of the beams and to assist in resisting seismic forces and lateral forces acting on the floor from general use, a bracing system should be considered. This is shown in the building plans shown in Figure 3.16, where bracing is provided by a mix of L102x102x6.4 braces and C150x12 braces.

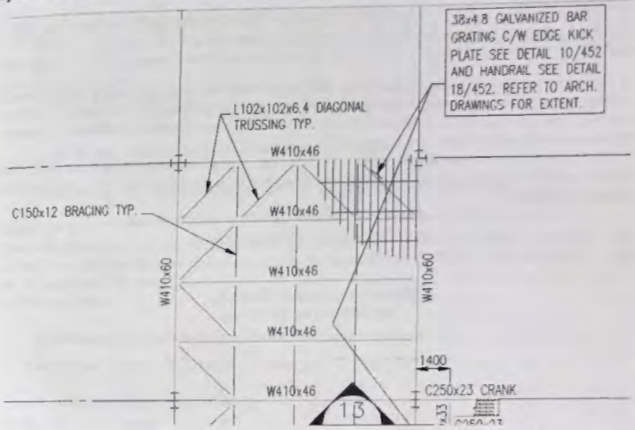


Figure 3.16. Bracing provided by mix of L102x102x6.4 braces and C150x12 braces

3.8 SHEAR RESISTANCE OF BEAMS

The shear resistance of beams is covered in S16 Clause 13.4. This clause gives the shear resistances both for thin-webbed beams with stiffeners added perpendicular to the axis of the beam and for beams without these stiffeners.

The results for beams without stiffeners can be derived from the equations that apply to the stiffened beams providing the following assumptions are made:

- a = distance between web stiffeners = large or infinite if no stiffeners.
- a/h = aspect ratio considered as being infinite when computing k_a and k_v
- k_a = aspect coefficient = 0. $\quad k_a = \frac{1}{\sqrt{1 + (a/h)^2}} = \frac{1}{\sqrt{1 + \infty^2}} = 0$
- k_v = shear buckling coefficient = 5.34 $\quad k_v = 5.34 + \frac{4}{(a/h)^2} = 5.34 + \frac{4}{\infty} = 5.34$

For members analyzed elastically (i.e., the normal way to do the analysis), the factored shear resistance, V_r, developed by the web of a flexural member having two flanges shall be taken as:

$$V_r = \emptyset A_w F_s$$

where:

A_w = shear area (d*w for rolled shapes h*w for WWF, 2*h*t for rectangular HSS).

F_s = ultimate shear stress determined as follows:

When $\frac{h}{w} \le \frac{1014}{\sqrt{F_y}}$ $F_s = 0.66F_y$ (For F_y = 345MPa, $\frac{1014}{\sqrt{F_y}} = \frac{1014}{\sqrt{345}} = 54.6$)

When $\frac{1014}{\sqrt{F_y}} < \frac{h}{w} \le \frac{1435}{\sqrt{F_y}}$ $F_s = \frac{670 * \sqrt{F_y}}{(h/w)}$ (For F_y = 345MPa, $\frac{1435}{\sqrt{F_y}} = \frac{1435}{\sqrt{345}} = 77.3$)

When $\frac{h}{w} > \frac{1435}{\sqrt{F_y}}$ $F_s = \frac{961200}{(h/w)^2}$

For those members that are analyzed plastically with redistribution of moments following development of plastic hinges, the shear capacity is reduced by 20%, as there will be both high shears and moments at those locations and reducing the permitted shear by 20% results in no reduction in the flexural capacity. For members with web stiffeners, see Clause 13.4.

For beams without web stiffeners, Figure 3.17 shows the factored resistance stress in the beam web. As with curves for bending capacity of unsupported beams and axial capacity of columns, there is a three part curve comprising:

- A flat portion where the material is fully plastic and cannot take more load.
- A portion at high slenderness where elastic buckling governs.
- A transition curve between the two.

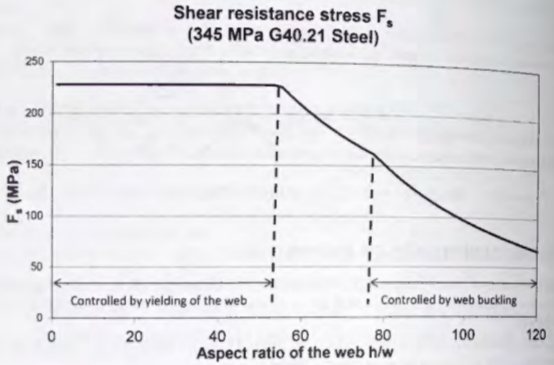


Figure 3.17. Factored resistance stress in the beam web

EXAMPLE—SHEAR DESIGN

Design the shear for a W410x39 beam with F_y = 345MPa

Solution 1:

The shear capacity can be determined from the beam selection tables (blue pages) of the *CISC Handbook* as V_r = 480kN

Solution 2:

Using the simplified provisions used in S16 for unstiffened beams.

The W410 x39 is a rolled shape and has the following properties:

d = 399mm; h = d–2t = 381mm; w = 6.4mm

h/w = 381/6.4 = 59.5

Web area = 6.4mm * 399mm = 2554mm² (Note that d*w is used for rolled shapes)

Examine where each of the three equations is appropriate:

The value of h/w of 59.5 is greater than 54.6 that if less would require the use of the first equation, but less than 77.3 that if greater would require use of the third equation, so we will therefore use the middle (transition) equation:

$$F_s = \frac{670 * \sqrt{F_y}}{(h/w)} = \frac{670 * \sqrt{345}}{(59.53)} = 209\,MPa$$

$V_r = Ø * A_w * Fs = 0.9 * 2554 \times 10^{-6} m^2 * 209{,}000 kN/m^2 = 481 kN$

(Again agrees with value in the beam selection tables of the *CISC Handbook*.)

Solution 3:

Using the equations in S16 for stiffened beams and realizing that with most beams there will be no vertical stiffeners added to increase shear capacity use the formulas for a stiffened beam, the shear resistance can be computed. (We would never do this, but it is included to illustrate that the answer does come out to the same value.)

The W410 x39 is a rolled beam and has the following properties:

d = 399mm; h = d–2t = 381mm; w = 6.4mm

a/h = Infinite as there are no stiffeners

h/w = 381/6.4 = 59.53

Web area = 6.4mm * 399mm = 2554mm²

k_n = 0 as there are no stiffeners

Calculate k_v: using the formula when a/h>1

$$K_v = 5.34 + \frac{4}{(a/h)^2} = 5.34 + \frac{4}{(Infinity)^2} = 5.34$$

Compute cut-off points appropriate for the yield stress of the steel:

$$439\sqrt{\frac{k_v}{F_y}} = 439\sqrt{\frac{5.34}{345}} = 54.62 \qquad 621\sqrt{\frac{k_v}{F_y}} = 621\sqrt{\frac{5.34}{Fy}} = 77.26$$

As h/w of 59.5 is > 54.6 and < 77.3, then $F_s = F_{cri}$

$$F_{cri} = 290\frac{\sqrt{F_y k_v}}{(h/w)} = 290\frac{\sqrt{345 * 5.34}}{(59.5)} = 209\,MPa$$

$V_r = Ø * A_w * Fs = 0.9 * 2554 \times 10^{-6} m^2 * 209{,}000 kN/m^2 = 481 kN$
(Agrees with value in the beam selection tables of the *CISC Handbook*.)

Shear is seldom a governing criterion where beam load is uniform for the selection of wide-flange beams. For wide-flange beams that support columns and for wide-flange girders with point loads near the ends, shear should be checked. Where design shear exceeds the shear that would be expected if the beam were loaded with the uniform load necessary to produce M_r, the shear to be used by the connection designer should be included on the drawings.

It is possible to prove that for most normal simple span beams it is not necessary to check for shear resistance. Take a simple span beam of length L and with uniform load W_f as shown in Figure 3.18.

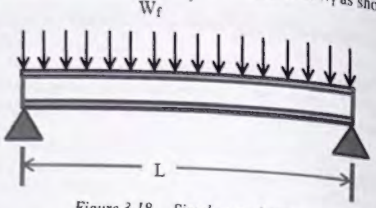


Figure 3.18. Simple span beam

The shear on this beam at the end is $V_f = \frac{W_f L}{2}$

The moment on the beam is $M_f = \frac{W_f L^2}{8}$

Manipulating this we get $M_f = \frac{W_f L^2}{8} = \left(\frac{W_f L}{2}\right)\left(\frac{L}{4}\right) = V_f\left(\frac{L}{4}\right)$

Further manipulation yields $\frac{M_f}{V_f} = \frac{L}{4}$

What we see from this calculation is that the shear on a uniformly loaded beam is a function of the span of the beam and, as the span increases, the beam design is more likely to be controlled by the bending resistance than it is by the shear resistance.

Our next step is to examine several beam sizes and see if there is relationship between the shear resistance of the beam and its moment resistance. We can then look at several commonly available W410 beams and compare their moment resistance as shown in Table 3.10.

Table 3.10. Commonly available W410 beams

	MR	VR	MR / VR
	(KNM)	(KN)	(M)
W410x39	227	480	0.47
W410x46	275	578	0.48
W410x54	326	619	0.53
W410x60	369	642	0.57
W410x67	422	739	0.57
W410x74	469	821	0.57
W410x85	537	931	0.58

Table 3.10 shows that the moment resistance rises faster than the shear resistance as the weight of the beam goes up and we are more likely to have the shear capacity controlling for the heavier beams than for the lighter beams at the same nominal depth.

If we look at a typical span-to-depth ratio of 20 for a floor beam, we would expect a W410x85 to span about 20*410 = 8.2m. With a yield moment of 537kNm, the factored shear at the point where the beam moment capacity is reached would be:

$$V_f = \frac{M_f 4}{L} = \frac{537kNm * 4}{8.2m} = 262kN$$

This represents 28% of the factored shear resistance of 931kNm and the design of the beam is controlled by moment capacity rather than by shear. If we repeat the above calculation for various beam spans, using the example of a W410x85 beam, and graph how much of the shear capacity is used when we the moment resistance is achieved, we get the graph shown in Figure 3.19.

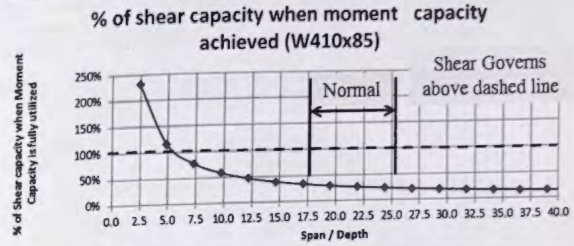


Figure 3.19. Percentage of shear capacity when moment capacity achieved

What we can see in Figure 3.19 is that, for a uniformly loaded simple span beam in the normal range of span-to-depth, the shear load is less than 50% of the factored shear resistance at the time that the factored moment is reached. This gives us a clear indication of regions where shear starts to govern the design:

- Stocky beams with span-to-depth ratios of 5 or less.
- Beams with point loads, particularly if the point load is near the end. This occurs during column transfers.
- Beams with thin webs such as those for WWFs.

While it has been shown that there are many cases for which shear will not govern the design of a beam, it is clearly the responsibility of the structural engineer to make sure that the shear capacity is sufficient to resist the design loads.

3.9 HOLES IN BEAM WEBS

Requests from the site for holes through the webs of beams often occur after the erection of the structure and often must be reviewed in a short timeframe. A computer program to assist in this task is of great value and sample outputs from such a program follow.

Beam holes are covered in S16 Clause 14.3.3, which states that circular holes are permitted in the web of beams without reinforcing provided ALL the following conditions are satisfied:

1) Hole is circular in shape (rectangular holes must be analyzed).
2) Beam section must be Class 1 or 2. (Class 3 and 4 beams must be analyzed.) The ability to provide holes in the webs of Class 1 and 2 beam sections is one advantage of avoiding Class 3 and 4 beams when designing floors and roofs.
3) Beam load must be uniformly distributed. (This eliminates beams with point loads so that girders picking up beams and beams or girders supporting joist loads would, in theory, be eliminated, but in practice the same methods are used to review holes through the webs of these beams provided the hole is located longitudinally a beam depth away from the point load.)
4) Section has an axis of symmetry in the plane of bending. (Wide-flange beam sections comply, but channels do not.)

nearer to the flanges or the beam ends must be analyzed.)

6) The longitudinal centre-to-centre spacing between adjacent openings is at least 2.5 times the diameter of the larger opening.

7) The maximum factored shear at the support is no greater than 50% of the factored shear resistance of the section. (This criterion will be easy to satisfy unless the beam is a welded wide flange section with a very thin web compared to its moment capacity, or in cases of girders picking up columns.)

While these seven criteria might seem to be restrictive, they are frequently satisfied when evaluating whether sprinkler lines or conduit can pass through the beam as shown in Figure 3.20.

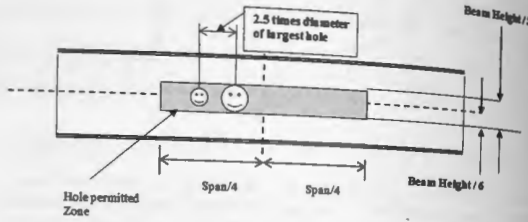


Figure 3.20. Circular holes permitted in beam design

For other cases, including the placement of rectangular holes, beam hole analysis is described in the *CISC Handbook*. The procedure includes worked examples and a flow chart if it is desired to program a computer program to perform the task. In general, beam holes are acceptable if certain conditions to satisfy stability are satisfied as well as conditions to satisfy strength requirements of the remaining web, including the requirement to provide rounded corners for the holes.

The method in the *CISC Handbook* applies only to Class 1 or Class 2 beams. If a Class 3 beam is encountered it would seem reasonable to perform the calculations outlined in the *Handbook* but using a yield point for the steel that is low enough to convert the Class 3 beam to a Class 2 beam. If the beam hole and the beam section work with the use of a theoretically lower yield point in the presence of the hole, it would be reasonable to accept the hole. In some cases it may be necessary to reinforce the hole by adding horizontal plates top and bottom or horizontal and vertical plates, as shown in Figure 3.21 and as described in Figure 3.22.

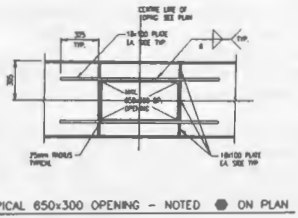
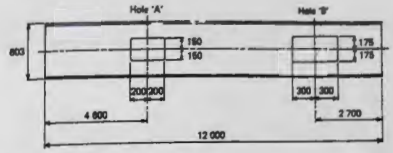


Figure 3.21. Reinforcing beam web openings by adding horizontal and vertical plates to stiffen the beam web



Check of unreinforced web hole following method in 1994 CISC handbook Page 5-154
Sample problem page 5-161 CISC Manual
Yield of Steel= 350 MPa
At center of Hole M= 596.KNm
Vh= 49. KN
Eccentricity of center of hole: e= 0.0 mm
Zero Eccentricity means hole centerline is same horizontal line as beam centerline
Hole is rectangular
Hole Dimensions Vertical = 300. mm
Horizontal=400. mm

Beam is W610 x 101
Beam Properties:
d=603 mm A= 13000. Ix=764. Zx=2900. Iy=30. Zy=464. J=781. Cw=2980.
Beam is Section Class: 1

Alpha1=.43 Alpha2=.43
As Eccentricity of hole is zero we expect Alpha1=Alpha2

Computs if V1/Vr ratio is acceptable:

V1/Vr= .276
Vr'=1097.KN V1= 302. KN
V(49 KN) is < V1 (302 KN) OK

Check if M1/Mr ratio is acceptable:

M1/Mr= 4151 Mr= 914 KNm M1= 379.
Mo/Mr=.9213 Mo=841.633 KNm C2=1.837
M(596KNm) is < Maximum M (767 KNm) OK

Check Web Stability:
Check: (2a/2H) + 6(2H/d) <= 5.6
2a / 2H = 400. / 300. = 1.3
6(2H/d)= 6 * 300./ 603. = 3.
(2a/2H) + 6(2H/d)= 4.32
Situation is satisfied

Vf < .67 Vr?
Vh= 49. KN .67*Vr'= 735. KN
Vf< .67 Vr Situation is OK
a/H to be less than 3
a / H = 200. mm /150. mm= 1.33
Situation is satisfied

Check Compression zone stability
OK if 2a <= 4dt
dt = d/2 - e - H
dt=603. / 2 - 0.0 - 300. / 2 = 152.
2a=Horiz. Dimension of Hole= 400. 4dt= 4 * 152. = 606.
Situation is satisfied as 2a <= 4dt

Note-Hole to have a corner radius at least equal to 16mm or twice web thickness
Beam Web =10.5 mm Twice Beam web= 21. mm
Beam Web requirements govern use 21 mm Radius at corners

Input to program

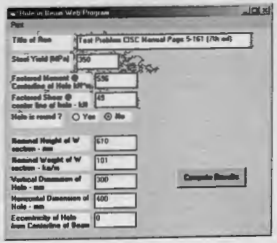


Figure 3.22. Sample input and output from beam hole analysis program

3.10 TORSIONAL CAPACITY OF WIDE-FLANGE BEAMS

Open sections such as wide-flange beams, channels, and angles are weak in torsion and tend to act in a manner similar to a group of individual plates, as shown in Figure 3.23.

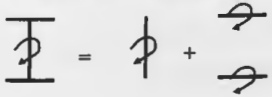


Figure 3.23. Torsional behaviour of open sections

This weakness results in two things:

1) It does not take much to twist the open section and resulting deflections may be excessive, especially for something cantilevering from the twisting beam.
2) The structural capacity of the section itself is low and it does not take much torsional moment to exceed the allowable stresses of an open section.

The St. Venant torsional constant of a plate is approximately $\frac{b*t^3}{3}$ where t is the smaller dimension.

EXAMPLE—DETERMINE TORSIONAL CONSTANT OF OPEN SECTION

Determine the approximate torsional constant for a W410x39 beam.

Solution 1:

The table of W-shapes in the *CISC Handbook* lists the torsional constant, J, as $111 \times 10^3 mm^4$.

Solution 2:

Use the equation for a plate and sum the properties for the flanges and web.

The flange width is 140mm and the flange thickness is 8.8mm.

The web height is 381mm and the web thickness is 6.4mm

$$J = \frac{140*8.8^3}{3} + \frac{381*6.4^3}{3} + \frac{140*8.8^3}{3} = 31802mm^3 + 33292mm^3 + 31802mm^3$$

$$J = 96.9 \times 10^3 mm^4$$

The results is 12.7% lower than the value of $111 \times 10^3 mm^4$ in the *CISC Handbook*, in part due to neglecting the presence of the fillets.

The torsional capacity of closed sections is much higher as the torsional constant is dependent on the area enclosed by the centreline of the section. Note that this is not the gross area of the section. It is, in the case of HSS sections, the area inside a line drawn at the centreline of the tube wall.

EXAMPLE—DETERMINE TORSIONAL CONSTANT OF CLOSED SECTION

Determine the approximate torsional constant for an HSS 203x203x6.4 section.

Solution 1:

The table of HSS shapes set out in Part 6 of the *CISC Handbook* lists the torsional constant *J* as $49300 \times 10^3 mm^4$. The weight of the 203x203x6.4 (38.4kg/m) is slightly less than that of the W410x39 (39kg/m), but its torsional constant is almost 450 times as great as the open section of the wide flange.

Solution 2:

Use the formula for the torsional constant of a closed section.

$$J = \frac{4(A_{enclosed})^2 t}{L_{enclosingPath}}$$

$A_{enclosed}$ = area enclosed by a line through the centre of the walls of the HSS

$A_{enclosed} \approx (203–6.4) * (203–6.4) = 38652mm^2$

$L_{enclosingPath}$ = length of line through the centreline of the plates (shown in Figure 3.24) that encloses the section $\approx 4 * (203–6.4) = 786mm$

t = thickness of wall 6.4mm

$$J = \frac{4(A_{enclosed})^2 t}{L_{enclosingPath}} = \frac{4*(38651mm)^2 * 6.4mm}{786mm} = 48634 \times 10^3 mm^4$$

The result is within 1.4% of that listed in the *CISC Handbook*.

EXAMPLE—DETERMINE TORSIONAL CONSTANT OF CLOSED SECTION

A 0.5kN load from a sign is attached at the centreline of a 10m long W410x39 beam. The load is eccentric to the beam by 750mm. Compute the approximate deflection at the centreline of the sign shown in Figure 3.25.

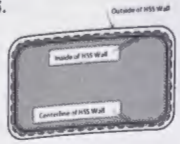
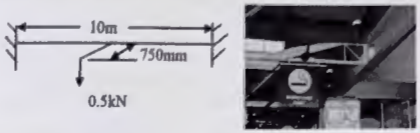


Figure 3.24. Centreline of HSS plate



a)

b)

Figure 3.25. a) Sign to be designed. b) Centreline calculation

The vertical bending deflection is:

$$\Delta = \frac{PL^3}{48EI} = \frac{0.5kN * (10m)^3}{48 * 200 * 126} * 10^3 mm/m = 0.41mm$$

As the load is located in the centre of the beam, the rotation at the centreline of the beam due to the torque from the eccentric load is computed by finding the rotation from half the torque acting on a beam of half the full length.

$$\phi = \frac{TL}{GJ} = \frac{(0.5kN * 0.75m/2) * 5m.}{77x10^6 kN/m^2 * 0.110x10^{-6} m^4} = 0.110 \text{ radians (6.3 degrees – very visible)}$$

Deflection at centre of sign = 0.110 radians * 750mm = 82.5mm

Total deflection at sign centreline = 0.4mm + 82.5mm = 82.9mm

Repeating these calculations using an HSS 203x203x6.4 gives, vertical deflection = 1.7mm + 0.185mm = 1.9mm. The deflection for the HSS-supported sign is approximately one fortieth that for the similar mass wide-flange section.

3.11 CANTILEVER BEAM STABILITY

Steel has a high strength-to-weight ratio and high capacity in tension and compression that makes it well-suited to carrying loads, but the buckling of beams and compression elements must be guarded against. Cantilever beams can be subject to lateral buckling, and this was dramatically illustrated in the failure of the Save-On Foods roof in Burnaby in 1989 where a roof with Gerber framing collapsed on opening day.

The Gerber framing system is popular for roofs where girders can be continuous over columns. The system can be viewed in many big box warehouses. The framing method is shown schematically in Figure 3.26.

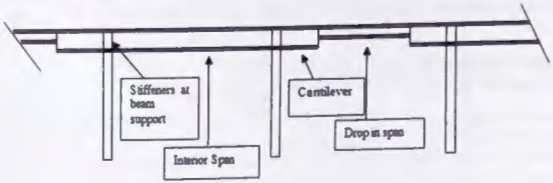


Figure 3.26. Gerber framing system

The advantages of this system are:

a) The interior span has negative moments at the columns, reducing the positive moment at midspan and this can reduce the size of the interior span girder.

b) The drop-in span girder can be smaller than it would be if it had to span the full distance between columns.

c) The negative moments on the interior span reduce the deflections on this beam.

The disadvantages of the system are:

d) The system requires more analysis and design time than does a series of simple span beams.

e) The system can be subject to lateral instabilities if the cantilever beam is not properly designed and detailed.

f) The system is only appropriate for use in roofs, as the beam cantilevers over the column. In multi-storey buildings the columns are usually continuous over more than one floor to avoid the work of column splices, and the beams on lower floors are therefore framed into the sides of the column rather than passing over it.

g)

NBCC Clause 4.1.6.3 requires that we examine two load cases:

Full snow load everywhere with appropriate C_s factors applied.

Use $C_s = 1.0$ everywhere, use pattern loading of 100% and 50% snow load to give the most unfavourable loading condition.

This is shown in Figure 3.27.

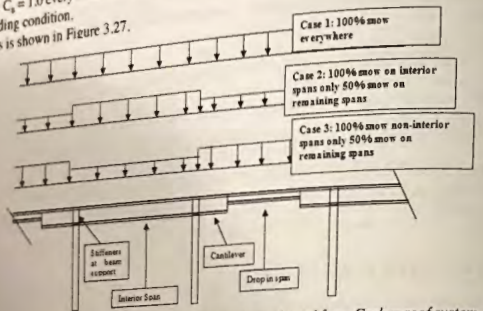


Figure 3.27. Variations in snow loads to be evaluated for a Gerber roof system

The cantilever should be sized with 50% snow load on the interior span, as this gives the longest unsupported effective length. The interior span should also be checked with 50% snow load on the cantilever spans. It is possible to have the configuration such that the interior span is under negative moment (bottom flange in compression) through its entire length. This is an undesirable situation and must be accounted for in the design by the use of appropriate unsupported lengths when sizing the beam.

3.12 DETERMINING THE UNSUPPORTED LENGTH OF THE CANTILEVER BEAM

The fundamental problem with cantilever beams is determining the appropriate unsupported length to be used in a stability analysis. The unsupported length appropriate for design will depend on the type of beam restraint at the supporting column and also at the cantilever tip. To some degree, it also depends on the backspan stiffness and loading. Some designers have expressed frustration at the lack of specific guidance in S16, but the complexity of the problem has made the establishment of simple, widely-applicable design rules extremely difficult. Nevertheless, with some background knowledge about assumptions and

limitations, existing published research can provide useful design guidance. In some cases, if the beam is poorly restrained at the column and poorly restrained at the tip of the cantilever, the effective length can be 7.5 times the cantilever length. In the Gerber system, the column would be described as the root of the cantilever, while the end of the cantilever supporting the drop-in span would be described as the tip of the cantilever. This clearly has a profound effect on the resistance of the beam. A figure from *Roof Framing with Cantilever (Gerber) Girders and Open Web Steel Joists* by the CISC that shows appropriate unsupported lengths to use shown in Figure 3.28. It should be noted that the booklet is no longer in production and other more recent materials on this issue must be acquired from several sources.

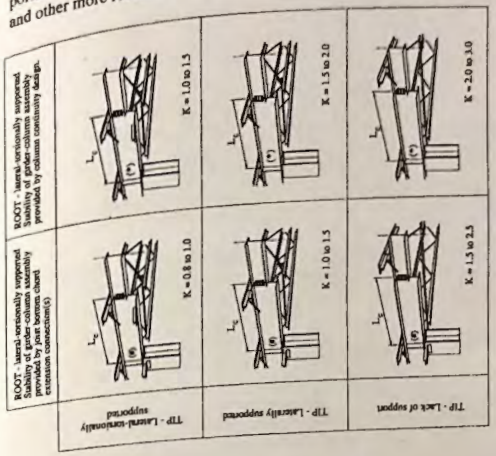
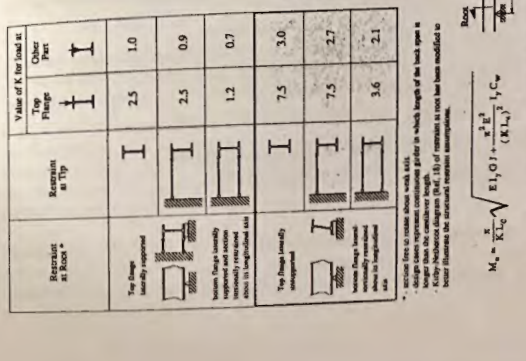


Fig. A2 Proposed Effective Length Factors (K) for Gerber-Cantilever Design

a)

Figure 3.28. Appropriate unsupported lengths: a) Fig. A1 Kirby-Nethercot Proposed Effective Length Factors. b) Fig. A2 Proposed Effective Length Factors (K) for Gerber-Cantilever Design
Source: Roof Framing with Cantilever (Gerber) Girders and Open Web Steel Joists (Canadian Institute of Steel Construction, July 1989).



$$M_u = \frac{\pi}{K L_c}\sqrt{E I_y G J + \frac{\pi^2 E^2}{(K L_c)^2} I_y C_w}$$

Fig. A1 Kirby-Nethercot Proposed Effective Length Factors (K)

b)

Figure 3.29 shows a figure from the *Guide to Stability Design Criteria for Metal Structures*, 5th edition, edited by Galambos. Unfortunately, this figure was dropped from the 6th edition. (Note that there are slight differences when compared to the information in Figure 3.28.)

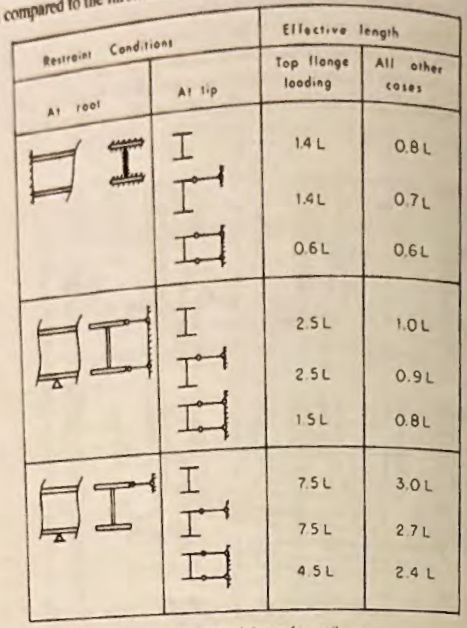


Fig. 5.11 Effective-length factors for cantilevers.

Figure 3.29. Effective-length factors for cantilevers
Source: Guide to Stability Design Criteria for Metal Structures, 5th edition. Edited by Theodore Galambos (John Wiley & Sons, 1998).

Four very important results can be deduced:

1) The beam should have lateral restraint at the column, provided by full height stiffeners and bottom chord extensions on the joist on column line.
2) The beam should be laterally restrained at the tip of the cantilever.
3) The correct unsupported length of the cantilever beam must be considered when sizing that beam.
4) The negative moment caused by the cantilever span must be considered if it travels too far into the interior span. In a poorly proportioned system, the entire interior span might be in negative moment with the compression flange unsupported over its entire length.

EXAMPLE—GERBER CANTILEVER BEAM EFFECTIVE LENGTH

If in Figure 3.30 the cantilever length is 2.0m, find the unbraced length.

Solution:
Examining Figure A2 from *Roof Framing with Cantilever (Gerber) Girders and Open Web Steel Joists* we find that, with no stiffener and no bottom cord extension at the column, no figure resembles the situation shown. Note that the framing details shown in the figure are no longer accepted practice.

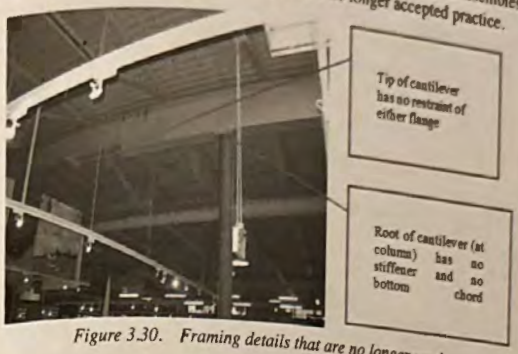


Figure 3.30. Framing details that are no longer used

Using Galambos we find the value is between 3.0L and 7.5L, depending on the loading condition. In our case, the load will be partly through the top flange and partly through the plate connecting the drop in beam. However in this case it would be appropriate to go with the factor of 7.5, as the top flange of the beam is not restrained directly at the column.

The unsupported length to be used in design (as shown in Figure 3.31) should therefore be 7.5*2m = 15m, which would significantly reduce the capacity of the beam.

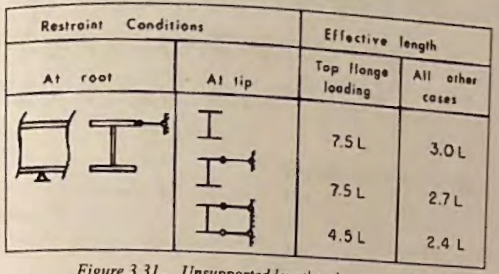


Figure 3.31. Unsupported length to be used in design
Source: Guide to Stability Design Criteria for Metal Structures, 5th edition. Edited by Theodore Galambos (John Wiley & Sons, 1998).

It might also be argued that in the example of Figure 3.30, the loading on the beam comes in through the shear plate to the adjacent beam and therefore is web loading (see figure from *Roof Framing with Cantilever (Gerber) Girders and Open Web Steel Joists*) and a K of 3.0 would be appropriate. However the consequences of cantilever beam failure are usually very dramatic and caution, and perhaps an independent analysis, should be used when picking the lower values of K.

EXAMPLE—GERBER CANTILEVER BEAM EFFECTIVE LENGTH

A Gerber cantilever beam is supported at the top and bottom flanges at the column using a joist bottom chord extension. At the cantilever only the top flange is supported. Of the load, 75% is applied to the beam

through the web connection to the drop in span and the remainder comes from the joist load at the top flange. If the cantilever length is 2.0m, find the unbraced length to be used in design.

Solution 1:
Examining Figure 3.28a (A1), we find the restraint at the root is such that the beam is restrained to top and bottom flanges. The tip has restraint to the top flange, K = 0.9 if loaded at web and K = 2.5 if loaded at top flange. If 75% of load is at web could do linear interpolation and get K = 1.46 Unbraced Length = K*L = 1.46*2.0m = 2.92m.

Solution 2:
Examining Figure 3.28b (A2), we find the restraint at the root is at the top and bottom flanges. With no joist bottom chord extension at the cantilever tip, K ranges 1.0 to 1.5, so use the more conservative value of 1.5. Unbraced Length = K*L = 1.5*2.0m = 3.0m.

From a philosophical standpoint it is desirable that stability not become a numbers game. Efforts should be made to provide the stabilizing elements and a conservative approach to selecting the unbraced length would not be unreasonable.



Figure 3.32. A well-detailed Gerber system has a full height stiffener at the column, a joist at the column line and that joist has a bottom chord extension

To examine the numeric effect of these stability requirements, let's start by looking at a two span floor beam, which has one span as a cantilever. The floor beam has normal live load, so we must consider patterned live loading in which the live load can be on both spans simultaneously or on any one span. The NBCC requires that for live load we consider 100% live load in any area and 0% live load in other areas that would relieve that load.

EXAMPLE—CANTILEVER BEAM DESIGN

A floor beam cantilevers over a girder, as shown in Figure 3.32. The tributary width is 1.75m. There is a 1.0kPa partition load and a 2.4kPa live load. Design the beam, taking account of pattern live loading and instability issues. The deflection at the tip of the beam should be a maximum of 2.25m / 180 = 12.5mm

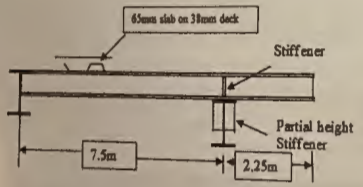


Figure 3.32. Floor beam cantilever over girder

Load Cases to evaluate:

Dead Load:
a) 0.9 * Dead Load everywhere
b) 1.25 * Dead load everywhere

We do not pattern dead load, with 1.25 dead on some spans and 0.9 dead on others, as it is assumed that the construction conditions are the same on all spans and if one span has extra dead load, then its neighbour will be in the same condition.

- 65mm topping on 38mm deck slab weight = 1.89kPa
- Partition load = 1.0kPa
- Estimate beam weight = 0.5kN/m
- Dead load = 1.75m * (1.89kPa + 1.0kPa) + 0.5kN/m
- Dead load = 5.6kN/m
- 1.25 dead load = 6.9kN/m
- 0.9 dead load = 5.0kN/m

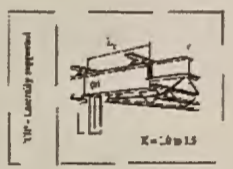


Figure 3.33. Laterally supported tip
Source: Guide to Stability Design Criteria for Metal Structures, 5th edition. Edited by Theodore Galambos (John Wiley & Sons, 1998).

Live Load:
- Service = 2.4kPa * 1.75m = 4.2kN/m
- Factored = 1.5 * 4.2kN/m = 6.3kN/m

Live load cases to evaluate:
a) (Load Factor = 1.5) * Live load everywhere.
b) (Load Factor = 1.5) * Live load on interior span
c) (Load Factor = 1.5) * Live load on cantilever span

Cantilever Moment:
- Factored Dead Load = 6.9kN/m
- Factored Live Load = 6.3kN/m
- Cantilever = 2.25m
- $M_f = wL^2 / 2 = (6.9 + 6.3)(2.25)^2 / 2$
- $M_f = 33.4$kNm

Unsupported length of cantilever:
- Root top and bottom flanges supported
- Tip top flange is supported
- Using *Roof Framing with Cantilever (Gerber) Girders and Open Web Steel Joists*, K = 1.5
- Using the *Guide to Stability Design Criteria for Metal Structures* (5th edition), K = 2.5

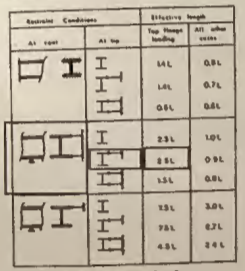


Figure 3.34. Effective-length factors for cantilevers
Source: Guide to Stability Design Criteria for Metal Structures, 5th edition. Edited by Theodore Galambos (John Wiley & Sons, 1998).

Using K = 2.5, the effective length, 5.6m, can be computed and a beam that satisfies the required moment can be found.

For a W360x33, M_r>38.0kNm>33.4kNm. OK.

Cantilever Span – use K=2.5

- K*L = 2.5*2.25m=5.6m
- M_f = 33.4 kNm

Designation	Vr kN	Ix 10^6mm^4	b mm	Lu mm	Mr <Lu	Factored Moment Resistance Unbraced Length (mm) 2000	6000	8000
W360x33	396	82.8	127	1600	168	-	38.0	
W250x39	384	60.1	147	2110	159	-	99.2	
W310x33	423	65.0	102	1320	140	143	31.7	
W200x46	300	45.2	203	3370	138	-	111.0	
W200x42	302	40.6	166	2610	137	-	91.9	
W310x31	294	67.2	164	2020	133	-	52.4	35.7
W310x28	360	54.3	102	1290	126	120	23.4	16.5
W200x36	256	34.1	165	2510	117	-	71.3	49.6
W250x28	341	40.0	102	1370	110	107	26.6	19.2
W200x31	275	31.4	134	1980	104	-	45.6	32.7

Figure 3.35. Cantilever span

Interior span:

- First kick – ignore moment from cantilever that reduces interior moment.
- $WL^2/8 = (6.9 + 6.3)*(7.5)^2/8 = 92.8$kNm
- Fully supported W360 x 33: M_r = 168kNm (OK – not worried about supported capacity; examine unsupported capacity)
- Examine with full live load on cantilever and no live load on interior span
- $M_{cantilever} = (5.0 + 6.3)(2.25)^2/2 = 28.6$kNm
- $M_{centreline} \sim (5.0) * (7.5)^2/8 - (28.6/2) = 20.9$kNm

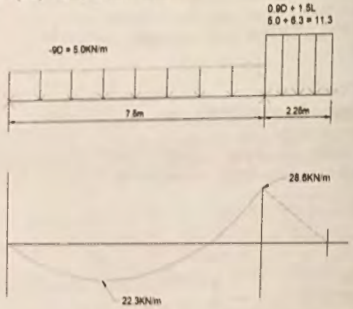


Figure 3.36. Interior span

An envelope of the bending moments possible is as follows:

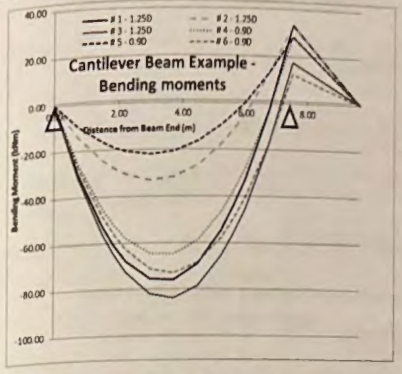


Figure 3.37. Cantilever beam example showing bending moments

We will use these moments to check the stability of the beam in the backspan region using the recommendations of the *Guide to Stability Design Criteria for Metal Structures*, 5th edition, as shown in Figure 3.38.

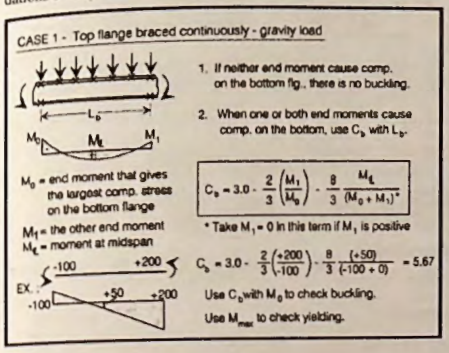


Figure 3.38. Top flange braced continuously for gravity load
Source: Guide to Stability Design Criteria for Metal Structures, 5th edition. Edited by Theodore Galambos (John Wiley & Sons, 1998).

C_b is a replacement for ω_2 in the M_u equation.

$$C_b = 3.0 - \frac{2}{3}\left(\frac{M_1}{M_0}\right) - \frac{8}{3}\frac{M_{centerline}}{(M_0 + M_1)}$$

$$C_b = 3.0 - \frac{2}{3}\left(\frac{0kNm}{28.6kNm}\right) - \left(\frac{8}{3}\right)\left(\frac{22.3kNm}{(-28.6+0)}\right) = 3.0 - 0 + 2.07 = 5.07$$

Look for $M_r' = 28.6\text{kNm} / 5.07 = 5.6\text{kNm}$ at $L_u = 7.5$m

M_r' of W360x33 = 28.4kNm (using linear interpolation between L_u = 7.0m and L_u = 8.0m values in beam selection tables (blue pages) of the *CISC Handbook*)

OK. W360x33 works for both interior and exterior spans.

Examine serviceability on interior span (using live load only):

- Determine live load deflection of interior span with no live load on cantilever
- Service live load = 4.2kN/m
- W360x33 I = 82.7mm⁴; Span = 7.5m
- Deflection = 10mm = 1/716 OK
- Interior span is OK

Examine serviceability on the cantilever span shown in Figure 3.39.

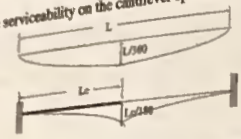


Figure 3.39. Serviceability on cantilever span

Downward Deflection = Cantilever deflection + Rotation at root * Cantilever length

$$\Delta = \frac{wL^4}{8EI} + (L_{cantilever}) * \frac{M_{support} * L_{backspan}}{3EI}$$

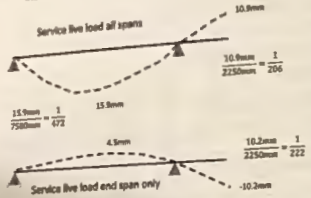


Figure 3.40. Downward deflection

$$\Delta = \frac{(4.2kN/m)2.25^4}{8*200*82.7} + (2.25m) * \frac{(4.2*2.25^2/2)*7.5m}{3*200*82.7}$$

LL Deflection = 0.001m + 0.003m = 0.004m

Span / Deflection = 4/2250 = 1/563

Want L/360 for twice cantilever length

Cantilever Length / 180 is acceptable (OK)

Check Live load on interior span only:

Rotation at end = 0.0046 radian

Cantilever Deflection = Theta*L

= 0.0046* 2250mm = 10.35mm

10.35/2250 = 1/217 OK (<1/180)

Use W360x33

Now let's extend this to a Gerber framing system. You will notice in this example that downward wind load is not combined with the snow loading. This is a reasonable assumption for addressing pattern snow loading as the pattern snow loading should itself be considered a rare loading. If the cases of full uniform snow loading were governing, then downward wind loading should be combined in the same way that we evaluated simple span roof beams when we discussed roof loading in Chapter 2.

EXAMPLE—GERBER ROOF DESIGN

Using data in Table 3.11 and Table 3.12, design a steel girder and infill girder for a cantilever beam system roof of a building located in an area where $S_s = 1.8$ and $S_r = 0.2$. Other loading assumptions are explained in the discussion that follows. The beam span is 12m, with 2m cantilevers at each end. The tributary width is 8.5m using joists estimated to weigh 0.1kN/m². The deck is 38mm deep, supported on OWSJs at 2000mm o/c. The overall roof is 50m x 36m in plan ($l_c = 46.1$m). There are no architectural restrictions as to beam depth. There is a bottom chord extension on the joist at the column and vertical stiffeners have been added in the beam at the column location. At the cantilever tip, only the top flange is supported.

For $l_c < 70$m, $C_b = 0.80$.

100% Roof Snow = {($C_a = 1.0$) * ($C_b = 0.8$) * ($S_s = 1.8$)} + ($S_r = 0.2$) = 1.64kPa

50% Roof Snow = 0.5 * 1.64kPa = 0.82kPa

Table 3.11. Roof loading in 100% snow region

Load Source	Dead (kPa)	Snow (kPa)	Wind (kPa)	Factored (kPa)
Vancouver Roof snow (Ss = 1.8 Sr = 0.2)		1.64		2.46
Interior suction and positive exterior			0.39	0.16
Roofing / Insulation / Ceiling allowance	0.90			1.13
Steel Deck	0.10			0.13
Total load for Deck Design	1.00	1.6	0.4	3.87
Suspended Mechanical / Electrical	0.20			0.25
Joists at 2000 o/c	0.10			0.13
Total load for Joist Design	1.30	1.6	0.4	4.24
W460x60 Girder at 8500 o/c	0.07			0.09
Total load for Girder Design	1.37	1.6	0.4	4.33

Table 3.12. Roof loading in 50% snow region

Load Source	Dead (kPa)	Snow (kPa)	Wind (kPa)	Factored (kPa)
Vancouver Roof 50% snow (Ss = 1.8 Sr = 0.2)		0.82		1.23
Interior suction and positive exterior			0.39	0.16
Roofing / Insulation / Ceiling allowance	0.90			1.13
Steel Deck	0.10			0.13
Total load for Deck Design	1.00	0.8	0.4	2.64
Suspended Mechanical / Electrical	0.20			0.25
Joists at 2000 o/c	0.10			0.13
Total load for Joist Design	1.30	0.8	0.4	3.01
W460x60 Girder at 8500 o/c	0.07			0.09
Total load for Girder Design	1.37	0.8	0.4	3.10

Reaction from joists in the 100% snow region:

P_f = (Joist Spacing = 2.0m) * (Joist Tributary = 8.5m) * 4.33kPa = 73.6kN

Reaction from Joists in the 50% snow region.

P_f = (Joist Spacing = 2.0m) * (Joist Tributary = 8.5m) * 3.10kPa = 52.7kN

Note that the weight of the girder is included directly in this calculation and is assumed to be part of the point load from the joists.

Note that a dead load factor of 1.25 is used throughout. We do not "pattern" dead load where alternating load factors of 1.25 and 0.9 are used for various spans.

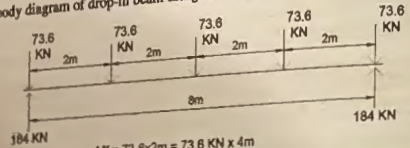
Note that even the areas of 0.5 snow are multiplied by a load factor of 1.5 as the same load factor is applied to the snow load everywhere on the structure.

Note also that the wind component is assumed to act on both the 100% snow region and the 50% snow region.

Design drop-in beam and find reactions each end:

Drop in span = 12m – 2m – 2m = 8m

Draw free-body diagram of drop-in beam using 100% snow loading as shown in Figure 3.41.



$M_f = 73.6x2m = \frac{73.6\ KN}{2} \times 4m$

M_f = 147.2 KN/m + 147.2 KN/m = 294.4 KN/m

Figure 3.41. Free-body diagram of drop-in beam

Factored Beam Moment = 294kNm

Span / 25 = 8000 / 25 = 320

Try W460x52 M_r' = 333kNm with top flange supported at 2m centres

Check factored net uplift pressure of 1.0kPa:

M_f = (1.0kPa*8.5m)*(8m)²/8 = 68kNm

W460x52 M_r' = 64kNm at L_u = 8.0m. Chose beam that has more uplift capacity

Try W410x54 M_r' = 86kNm at L_u = 8.0m. OK.

W410x54 has M_r' = 326kNm at L_u = 2.0m. OK (>294kNm) for loads from 100% snow loading

Compute deflection under service live load (ignore the wind and use 0.9 times the snow load. The 0.9 is the SLS importance factor for snow).

$$\Delta = \frac{5*W_f*L^4}{384*E*I} = \frac{5*(1.64kPa*0.9*8.5m)*8.0^4*1000}{384*200*186} = 18mm = L/444$$

RESULT:
For drop in span, useW410 x 54.

Design the beam that is continuous over the columns

The loads from each joist pair are the same as for the drop-in beam, namely:

100% snow region 73.6kN

50% snow region 52.7kN

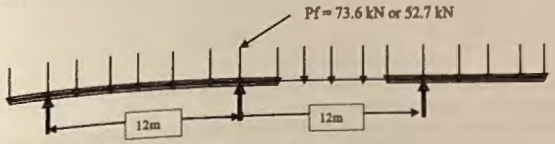


Figure 3.42. Joist pair

Compute cantilever moments

The maximum moment on the cantilever beam can be found by having the 100% snow loading on the drop-in beam. The results at the cantilever beam will be the same if the interior span is under 100% or 50% snow, but we will figure the cantilever moment as though there is 100% snow everywhere.

Maximum cantilever tip moment is computed as = (184kN) * 2m = 368kNm

We need to determine the value of "K" the unsupported length factor for the cantilever. The conditions are that there is a support to the cantilever at the top flange from the joist at the tip of the cantilever and a joist bottom chord extension and stiffener at the column.

Figure 3.43 shows laterally supported cantilever tips.

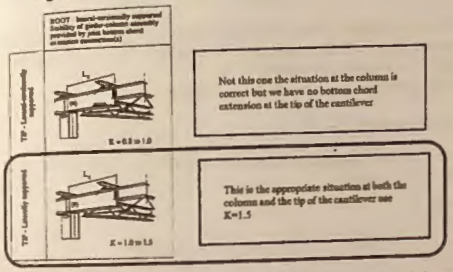


Figure 3.43. Laterally supported cantilever tips
Source: Figure A2 from *Roof Framing with Cantilever (Gerber) Girders and Open Web Steel Joists* (Canadian Institute of Steel Construction, July 1989).

It might be argued why not use the K = 1.0, but with stability it is probably better to lean toward the conservative end in these factors. $K*L_u = 1.5*2.0m = 3.0m$

Evaluate through *Guide to Stability Design Criteria for Metal Structures*, 5th edition, edited by Galambos, as shown in Figure 3.44.

Restraint Conditions		Effective length	
At root	At tip	Top flange loading	All other cases
		1.4L	0.8L
		1.4L	0.7L
		0.6L	0.6L
		2.5L	1.0L
		2.5L	0.9L
		1.5L	0.8L

Figure 3.44. Restraint conditions
Source: Guide to Stability Design Criteria for Metal Structures, 5th edition. Edited by Theodore Galambos (John Wiley & Sons, 1998).

The root is supported at top and bottom flanges (by the joist) and the bottom chord extension, while the tip is supported only at the top chord by the joist.

Using this method, we find we have a K of between 0.9L and 2.5L, depending on our loading conditions.

In our case, we have 73.6kN from the joist at the tip of the cantilever and 110kN from the shear plate connecting the drop in span for a total of 184kN. We therefore have about 40% of our load as top flange loading and 60% as other. Taking a linear interpolation assuming 100% top flange loading gives K = 2.5 and 0% gives K = 0.9 we get K = 1.54.

As shown in Figure 3.45, we have worked out K by two methods and both give about 1.5. Use K = 1.5.

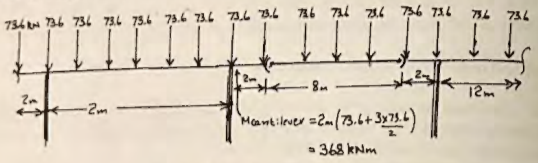


Figure 3.45. Cantilever moment

For M_r = 368kNm, ignoring the unsupported length we can get by with W460x52 with M_r = 397kNm but at K*L_u = 3.0m the M_r' of this beam is only 269kNm. Looking for a beam with depth approximately 12000/25 = 480mm, try W460x74 at K*L_u = 4.0m M_r' = 484kNm

We have checked the cantilever moments; we must now check the moments in the back-span.

Designing the cantilever back-span beam

Free body diagram (as shown in Figure 3.46) to give largest positive moment on interior span (100% snow on the interior span and 50% snow on the drop in span:

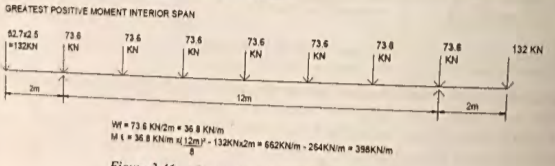


Figure 3.46. Greatest positive moment interior span

As a first attempt, check the W460x74 we have chosen from the cantilever moment:
M_r' = 512kNm at L_u = 2.0m (OK > 398kNm). The 2m unbraced length represents the spacing between the joists.

Free body diagram (as shown in Figure 3.47) under 100% snow on drop-in beam and 50% snow on interior span:

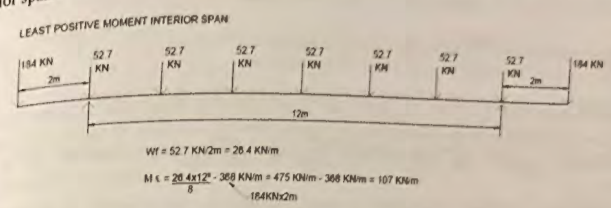


Figure 3.47. Least positive moment interior span

Resulting values of moment at column and middle of interior span is shown in Table 3.13.

Table 3.13. Moment at column and middle of interior span

Snow Exterior Span	Snow Interior Span	Cantilever Span Moment at Column	Moment Middle of Interior Span
100% (36.8kN/m)	100% (36.8kN/m)	368kNm	294kNm
100% (36.8kN/m)	50% (26.4kN/m)	368kNm	107kNm
50% (26.4kN/m)	100% (36.8kN/m)	264kN	398kNm

Note that under all circumstances we have positive moment on the interior span and the bottom flange is in tension over at least some of the length. It would be undesirable to have the negative moment go from one side of the beam to the other with the bottom flange in compression throughout its length.

The resulting moment envelopes can be plotted as shown in Figure 3.48.

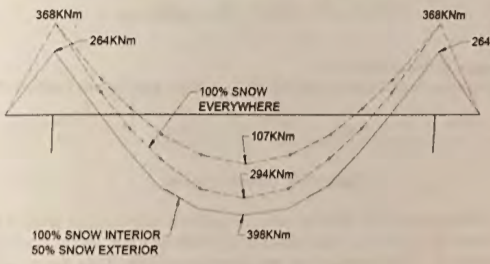


Figure 3.48. Moment envelopes

There is a considerable range of the location of the inflection point and it is very important to note that an inflection point is not a point of lateral restraint unless both flanges are supported at that point.

For a Class 1 or 2 beam, we would get:

$$M_r = \phi M_n = \phi \frac{C_b \pi}{L} \sqrt{EI_y GJ + \left(\frac{\pi E}{L}\right)^2 I_y C_w}$$

We must also check if M_u is greater than $0.67 * M_p$ in which case:

$$M_r = 1.15 \phi Mp \left(1 - \frac{0.28 Mp}{Mu}\right) \leq \phi Mp$$

Continuing our example, calculate C_b for uniform snow load (100% snow all spans):

$$C_b = 3.0 - \frac{2}{3}\left(\frac{M_1}{M_o}\right) - \frac{8}{3}\frac{M_{centerline}}{(M_o + M_1)} = 3.0 - \frac{2}{3}\left(\frac{-368}{-368}\right) - \frac{8}{3}\frac{294}{(-368 - 368)} = 3 - 0.67 + 1.065 = 3.398$$

Repeating this for the other two loadings that result from the snow pattern loading we get the results shown in Table 3.14.

Table 3.14. Snow pattern loading results

CASE	C_b	M_1	M_1 / C_b
100% snow everywhere	3.398	368kNm	108kNm
50% - 100% - 56%	4.34	264kNm	61kNm
100% - 50% - 100%	2.72	368kNm	135kNm

For a trial section we will now go to the beam selection tables (blue pages) of the *CISC Handbook* and find a W460 beam that has an M_r' of 135kNm at $L_u = 12m$.

This is satisfied by a W460x89:

$$M_r = \phi Mu = \phi \frac{C_b \pi}{L} \sqrt{EI_y GJ + \left(\frac{\pi E}{L}\right)^2 I_y C_w}$$

$$M_r = 0.9 * \frac{2.72 * \pi}{12000} * \sqrt{200 * 20.9x10^6 * 77 * 907x10^3 + \left(\frac{\pi * 200}{12000}\right)^2 * 20.9x10^6 * 1040x10^9}$$

$$M_r = 6.41x10^{-4} * \sqrt{29.19x10^{16} + 5.95x10^{16}} = 380kNm \quad \text{OK} - (>368\text{kNm})$$

The resulting design is shown in Figure 3.49.

Requires joist bottom chord extensions and full height stiffener at all column locations (Typical)

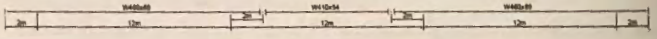


Figure 3.49. Resulting design

The design would require checks of deflection and some checks on wind uplift to finish. In this case the negative moment and the larger $K*L_u$ for the cantilever governed the size of the cantilever portion of the beam and the beam itself was governed by back-span slenderness considerations. A more efficient design might be achieved by adding bottom chord extensions at the cantilever tip and midspan of the interior to lower the unsupported length at the cantilever and the back-span. Reducing the 12m span should also be considered.

$$\text{Average weight of girders} = \frac{16m * 89kg/m + 8m * 54kg/m}{24m} = 77kg/m$$

If we were using a simple span system we would have:

$M_f = \frac{36.8kN/m * (10m)^2}{8} = 662kNm$, requiring a W610x82 beam to satisfy. The Gerber framing system therefore saves an average of 82kg/m – 77kg/m = 5kg/m.

It is important when using the Gerber framing system for beam design that the various stability conditions and patterned loading be evaluated. If the design relies on supports from bottom chord extensions and beam stiffeners at column locations, these should be called up on the drawings such that they are included in the fabricator's base price.

CHAPTER 4
COMPOSITE STEEL BEAMS

Up to this point we have considered the steel frame as if it were acting alone. The concrete that is an essential part of the floor system was just considered as dead load from the standpoint of the steel beams. In this chapter, we will consider the concrete slab as acting as a compression flange for the beams provided we manage to connect the two together in a way that invokes this behaviour. This can be an advantage in the design, as it may permit the use of smaller beams, but it does involve extra design and construction steps that makes the system more suited to beams or girders with high live loads.

Composite beams require three components:

1) a steel beam,
2) a concrete slab, and
3) mechanical connections between the slab and the steel beam.

The connector must directly engage the concrete to transfer shear between the concrete slab and the steel beam. A typical connector satisfying these requirements would be a Nelson stud welded to the beam after the deck has been placed. Systems where concrete on metal deck is attached to the steel beam only by deck puddle welding are not considered to be composite beams for strength or deflection purposes, but are considered to be composite for vibration analysis.

Composite beams are typically designed to be constructed using unshored construction. The non-composite steel beam acting alone, therefore, must carry its self-weight, the weight of deck and wet concrete, and the live load due to construction activity. (WorkSafeBC requires the design live load due to construction activity to be taken as a minimum of 1.9kPa [40 psf]; see the discussion on steel deck design.)

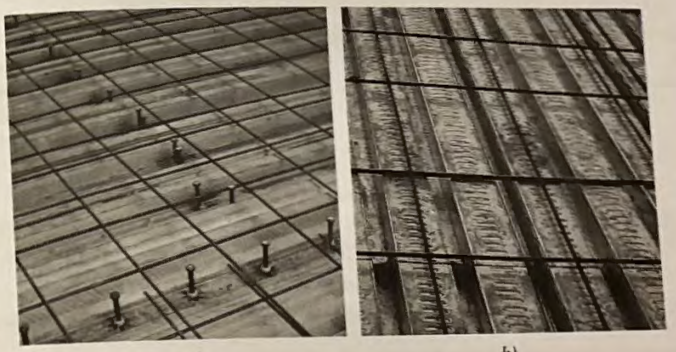
4.1 ADVANTAGES AND DISADVANTAGES OF COMPOSITE CONSTRUCTION

For the cost of adding some shear connectors, a reduction in the cost of the weight of the steel beam can be achieved. Depending on the relative costs of the stud installation and the price of the steel beam, this possibly reduces the construction cost. In some cases—particularly those with a high live load relative to the dead load—it is possible to get a lower structural depth than would be possible with a non-composite system.

It is important for the designer also to know what the disadvantages of composite construction are such that any drawbacks can be minimized in the design.

- Addition of the Nelson studs occurs after the deck is installed, but prior to placement of reinforcing steel for slab concrete. This causes an extra step in the field construction process, as well as specialized welding equipment.
- For the small deflections associated with live load due to walking activities, steel floor systems designed using the non-composite procedures (see Chapter 3) act as though they were composite from a vibration standpoint. While this composite behaviour cannot be used for strength analysis, it does reduce the vibration issues associated with human activity. As the steel beam size will be smaller with a composite design, the deflections and accelerations due to walking loads will be greater and the perception of vibration will be greater. Floors designed using composite beams are more likely to have vibration problems than those using non-composite floor designs.
- Additional costs are incurred in placing the shear studs, and it may be that the increase in cost from placing the shear studs exceeds the savings from a reduction in steel beam weight. The savings in the steel beam weight are just a material cost saving and the detailing of the beam; fabrication costs and erection costs will be similar for the beam with studs and without studs. In other words, to put a 6m long W410x39 in place instead of a 6m W410x46 involves many of the same detailing, fabrication and erection activities for the fabricator and erector, but saves 6m x (46kg/m – 39kg/m) = 42kg of material per beam. The actual material cost of the beam only represents about 30% of the cost of the in-place beam, with other costs being fabrication, erection, and detailing.
- To provide sufficient concrete cover for the studs, the steel deck needs to be a profile that is suitable for placement of studs. These decks will typically require greater concrete volume than composite deck profiles, resulting in more weight to be carried by the structure and additional concrete costs.

Figure 4.1 shows two floor decks.



a) b)

Figure 4.1. Two floor decks.
a) A floor system where the beams are composite with the slab. Note the addition of studs for composite action in both beams and girders.
b) A floor deck that is not connected to the beams by studs

Note that the deck in Figure 4.1a has narrower flutes and uses less concrete, as there is no requirement to provide cover to the Nelson stud. Before concrete placement, the reinforcing steel on the deck in Figure 4.1a will need to be chaired so that the bars are closer to the centre of the slab and to provide concrete cover to all sides of the bars, which will help the fire rating as well as the ability of the bars to resist cracking in the slab.

Figure 4.2 shows the resulting composite behaviour. Figure 4.3 shows two floor decks, one for use with a composite beam system and the other for use with beams that are not composite with the slab system.

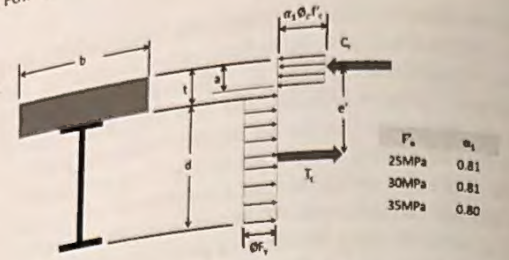


Figure 4.2. Resulting composite behaviour – instead of the beam acting as a flexural element it is assumed to be the tension element with the concrete acting as the compression element

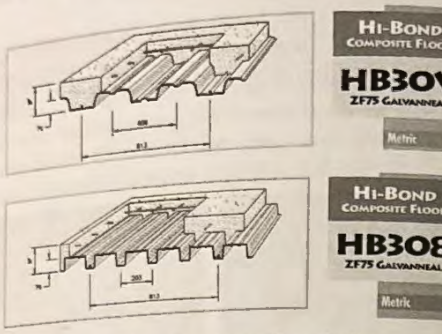


Figure 4.3 Composite floor decks:
a) Floor deck for use with composite beam system. For a 65mm topping on 76mm deck, this would take 0.099m³/m² of deck area and weigh 2.38kPa.
b) Floor deck for use with beams that are not composite with the slab system. For a 65mm topping on 76mm deck, this would take 0.087m³/m² of deck area and weigh 2.14kPa.

4.2 COMPOSITE STEEL ASPECTS FROM THE DESIGNER'S STANDPOINT

The following are general guidelines for the design of composite steel decks:

- Composite steel construction requires more design time and detailing effort than is required for non-composite construction.
- Installation of the Nelson studs should be inspected to verify that the correct size and spacing have been achieved, and that the weld quality is adequate. This may be performed at the time of reinforcing inspection.
- Spacing of Nelson studs, and other information necessary for construction, must be calculated and is additional information that must be placed on the drawings, increasing the work and costs for the consultant.
- Given the span of the beam and its size, the beam shear that is used for connection design of non-composite beams can be quickly determined from tables in the *CISC Handbook*. The structural design notes will state that the fabricator is to refer to these tables when designing the beam connections. The composite beam carries a greater factored load than the non-composite beam, and the shear for connection design must be specified more directly.

4.3 DESIGN PROCEDURES

The following are procedures for the design of composite steel decks:

1) Deck-slab thickness is determined based on deck procedures.
2) Beam span and spacing is selected to match floor layout and deck conditions.
3) Determine minimum beam size required to support all loads that occur prior to strength gain in the concrete slab, including live load from construction activities and the wet weight of the concrete.
4) Verify that the composite section is sufficient for factored loads expected after the concrete slab has hardened, including the live load from the use and occupancy of the floor and the self-weight of the concrete slab system.
5) Verify that the shear capacity of the section is sufficient; the beam web is assumed to carry the entire shear for the composite section. The V_r can be found from the beam resistance tables set out in the beam selection tables (blue pages) of the *CISC Handbook* and from first principles.
6) Determine the size and spacing of the shear studs necessary to provide the shear transfer for composite behaviour.
7) Review the behaviour of the composite section under specified loads, including the effect of concrete shrinkage.

Guidelines for the design of composite steel floors using wide-flange beams for buildings are as follows. These guidelines refer to beams with negligible axial load.

Similar to regular beam design, the wide-flange beams in the system are designed using 350W steel. While many steel manuals are available with composite beam tables using 300W steel, users should make sure that the tables they are using are appropriate for 350W steel.

WorkSafeBC interprets the beam as being part of the concrete falsework and requires the beam capacity to be sufficient to support a factored load as follows:

1) 1.25 times the weight of beam and deck supported, plus
2) 1.5 times the weight of the concrete supported (concrete assumed to be live load), plus
3) 1.5 times the live load of construction activity taken as 1.9kPa (40 psf).

The greatest savings from using a composite beam are achieved in those instances where the live load on the composite system is high. There will be no savings if the composite beam system carries a live load equal to the construction live load, as the non-composite beam must be designed for this load. For this reason composite beams are often used in bridge construction where the live load can be as high as 12kPa.

The properties of the composite beam can be determined by hand, by computer program, or by reference to tables such as those listed in the *CISC Handbook*.

When computing properties by hand, the effect of the reduced Young's modulus for concrete must be taken into account. The modular ratio is the ratio of Young's modulus of the steel to Young's modulus of the concrete and, while this modular ratio varies depending on the strength of concrete, it will typically be around 8 or 9 for typical strengths of concrete used.

$$n = \frac{E_{steel}}{E_{concrete}} = \frac{200000\,MPa}{4500\sqrt{F'c}} = \frac{200000}{4500\sqrt{25}} = 8.9$$

The effective slab width for composite design is half the distance to the adjacent beams or the span of the beam divided by 4, whichever is less.

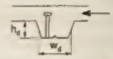
Capacity of the shear studs is given in the Nelson Stud catalogue or in Table 5-3 or 5-4 of the *CISC Handbook*. (Note that Table 5-2 is for solid slabs or slabs with ribs parallel to the beam, which would only be appropriate for a girder.) Use imperial-sized Nelson studs, as these are more readily available. In interpreting Table 5-3, normal weight concrete is 2300kg/m³. The 1850kg/m³ is lightweight concrete and is seldom used due to its limited availability (aggregates must be ordered specially), increased costs and difficulty in workability. Use of 25MPa concrete for composite construction is common. W_d = average width of deck flute; refer to the deck catalogue. It may be necessary to specify a deck that has a profile

that will suit the requirements of shear studs. As can be seen from Table 5-3, it is necessary to have a width of deck valley at least 1.4 times the height of the deck. For 75mm deck, we would therefore require a valley width of 105mm. The table also assumes that the nominal ultimate strength of the stud material is 450MPa. While this strength is common today, older structures may utilize 415MPa studs. If these properties are satisfied, the factored shear capacity of the stud is as shown in Table 4.1.

Table 4.1. Factored resistance of shear studs

Table 5-3

Factored Resistance of Shear Studs
Ribs Perpendicular to Beam
3/4" (19 mm) Diameter Studs
75 mm or 38 mm-Deep Steel Deck



Deck		Stud connector(s)				Pull-out area	Factored shear resistance of stud(s), q_r (kN)					
h_r	$\frac{w_r}{h_r}$	Dia.	Length	n_r	Edge distance	A_p	f'_c (γ_c = 2300 kg/m³)			f'_c (γ_c = 1850 kg/m³)		
mm		mm	mm		mm	10³ mm²	20 MPa	25 MPa	30 MPa	20 MPa	25 MPa	30 MPa
75	2.4	3/4"	115	1	Int.	52.0	85.2	72.9	79.8	55.4	61.9	67.8
		(19)		1	65	49.7	51.8	57.0	62.5	43.4	48.5	53.1
				1	38	33.9	42.5	47.5	52.1	36.1	40.4	44.2
				2	Int.	69.2	86.7	96.9	106	73.7	82.4	90.3
			150	1	Int.	82.7	74.6	86.7	94.1	63.4	73.7	83.4
				1	65	60.7	74.6	84.9	93.0	63.4	72.2	79.1
				1	38	51.8	84.8	72.5	79.4	55.1	61.6	67.5
				2	Int.	105	132	147	161	112	125	137

Stud shear capacity assuming one stud per rib (n = 1) and 65mm cover from the stud to the edge of the slab gives a factored resistance of 57kN / stud. For 75mm deck, it is also assumed that the studs are placed off-centre in the flutes toward the nearer support.

Note that Table 5-3 assumes that the beam is running perpendicular to the deck flutes. This is the typical case for repetitive floor beams, but not the case for girders picking up repetitive elements. If we were using a girder where the shear is parallel to the deck flutes, then we would use Table 5-2 in the *CISC Handbook*.

EXAMPLE—DESIGN OF SIMPLE SPAN COMPOSITE BEAM

Design a steel beam located in a hotel lobby floor using both non-composite and composite beam procedures. Beam span is 8.75m and the tributary width is 3.0m. The steel deck is 75mm deep with a 65mm thick topping.

First, examine as a non-composite beam:

DETERMINE LOADING ON AREA BASIS			
Load Type	**Service**	**Load Factor**	**Factored**
Assembly Live load	4.8	1.5	7.2
Partition load / finishes	1.0	1.25	1.3
Steel Deck	0.1	1.25	0.1
65 topping on 75 deck	2.14	1.25	2.7
W410 beam at 2750 o/c	0.2	1.25	0.3
	8.24		**11.6**

Factored beam load = Factored area loading * Tributary width
= 11.6kN/m² *3.0m = 34.8kN/m

$$\text{Factored beam moment} = \frac{WL^2}{8} = \frac{34.8kN/m*(8.75m)^2}{8} = 333kNm$$

$$\text{Factored beam shear} = \frac{WL}{2} = \frac{34.8kN/m*8.75m}{2} = 152kN$$

Span = 8750 / 20 = 438. Say, W460.

Try W460x52. M_r = 338kNm with top flange fully supported.

Compute deflection under service live load

$$\text{LL deflection} = \Delta = \frac{5*W_l*L^4}{384*E*I} = \frac{5*(4.8KPa*3.0m)*8.75^4*1000}{384*200*212} = 25.9mm = L/337$$

No go: Require I = (212x10⁻⁶m⁴) x (360/337) = 226x10⁻⁶m⁴

Use W460x60 with moment of inertia of 255x10⁻⁶m⁴.

Strength and deflection criteria are satisfied by W460x60 as a non-composite beam.

Second, examine as a composite beam:

The floor area loading goes up as the concrete weight on the deck increases.

DETERMINE POST-COMPOSITE LOADING ON AREA BASIS:			
Load Type	**Service**	**Load Factor**	**Factored**
Assembly Live load	4.8	1.5	7.2
Partition load / finishes	1.0	1.25	1.3
Steel Deck	0.1	1.25	0.1
65 topping on 75 deck	2.4	1.25	3.0
W410 beam at 2750 o/c	0.2	1.25	0.3
	8.5		**11.9**

Factored beam load = Factored area loading * Tributary width =
= 11.9kN/m² *3.0m = 35.7kN/m

$$\text{Factored beam moment} = \frac{WL^2}{8} = \frac{35.7kN/m*(8.75m)^2}{8} = 342kNm$$

The first step to examining as a composite beam is to determine the minimum size of beam necessary to support the weight of the wet concrete without shoring. If we use the WCB criteria of treating the wet concrete as a live load with a load factor of 1.5 with the requirement of a 1.9kPa live load with a load factor of 1.5, we would have the following loading criteria on the beam acting as a form.

DETERMINE PRE-COMPOSITE LOADING ON AREA BASIS:			
Load Type	**Service**	**Load Factor**	**Factored**
Construction Live load	1.9	1.5	2.9
Steel Deck	0.1	1.25	0.1
65 topping on 75 deck	2.4	1.50	3.6
W410 beam at 2750 o/c	0.2	1.25	0.3
	4.6		**6.9**

Factored beam load = Factored area loading * Tributary width

= 6.9kN/m² *3.0m = 20.7kN/m

$$\text{Factored beam moment} = \frac{WL^2}{8} = \frac{20.7kN/m*(8.75m)^2}{8} = 198kNm$$

Try W410 x39. M_r = 227kNm

The factored moment the beam is to be designed for has reduced from 333kNm to 198kNm – a reduction of 40%. The least weight of beam that will satisfy this moment is a W410x39 with an M_r of 227kNm. We therefore have the possibility of saving 60 – 39 = 21kg/m at the expense of adding shear studs if we can make this system work.

Compute deflection under the weight of wet concrete:

$$\Delta = \frac{5*W_l*L^4}{384*E*I} = \frac{5*(2.4kPa*3.0m)*8.75^4*1000}{384*200*126} = 19.6mm = L/437 \quad \text{(OK)}$$

Many designers will choose to camber this beam by 20mm such that the soffit of the beam will be level after concrete placement and so that additional concrete is not required to be added to make the floor flat. It is common also to include any dead load applied prior to the concrete curing into the camber calculation. The camber requirements must be shown on tender drawings.

Check the shear capacity of the proposed beam:

As the beam web is designed to carry the entire shear regardless of the beam being non-composite or composite, we must check the shear capacity of the beam proposed to be the composite beam. Our proposed W410x39 has a shear capacity of 480kN, exceeding the requirement of 152kN determined for the composite beam.

Examine the composite properties:

Our effective slab width is given by S16 Clause 17.4.1 as the lesser of the centre-to-centre spacing of the beams and the beam span divided by 4. Our effective slab width is therefore the lesser of:

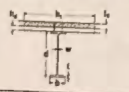
Centre-to-centre of beams = 3000mm

Beam span / 4 = 8750 / 4 = 2190mm

Referring to the table in the *CISC Handbook* (reproduced, in part, in Table 4.2), we see that for a 75mm deck on 65mm slab, with 2000mm effective slab width, the factored moment resistance, M_{rc}, in the final condition is 380kNm, provided we have a minimum of 40% shear connection.

Table 4.2. Portion of composite beams trial selection table

G40.21 350W
ASTM A992
A572 Grade 50
f'_c = 25 MPa



COMPOSITE BEAMS
Trial Selection Table
75 mm Deck with 65 mm Slab
ϕ = 0.90, ϕ_c = 0.65

Steel section	b_f	M_{rc} (kN·m) for % shear connection			Composite					Non-composite					
					Q_r	I_t	S_t	I_b			Unbraced condition			
	mm	100%	70%	40%	(kN)	10^6 mm^4	10^3 mm^3	10^6 mm^4			L' mm	M_r' kN·m	L' mm	M_r' kN·m
W410x39	3 000	496	445	383	1 550	521	1 150	415	M_r	227	2 000	216	7 000	44.1
W16x26	2 000	481	437	380	1 550	496	1 130	367	V_r	480	3 000	166	8 000	36.8
b = 140	1 500	447	412	360	1 550	458	1 110	333	I_x	1 750	4 000	105	9 000	31.3
t = 8.6	1 000	399	370	321	858	414	1 070	287	I_y	127	5 000	73.1	10 000	27.4
d = 399	500	333	307	275	429	332	998	223	S_x	634	6 000	55.2	11 000	24.3

Determining the shear stud connection requirements

For full shear connection the studs must develop the lesser of:

The tension capacity of the beam: $\phi A_s F_y$, and

The compression capacity of the concrete: $\alpha_1 \phi_c b_s t_c f'_c$

To be a considered to be composite and use the composite properties, we are required by Clause 17.9.4 to connect for a minimum of 40% of the full shear connection when using composite properties for strength and 25% of the shear connection when using the composite properties for deflection. This force must develop between the end of the beam and its point of maximum moment (typically midspan); and this force must be developed at each end.

In our case $\phi A_s F_y$ = 0.9 * 4990 x 10^{-6}m^2 * 350,000kN/m^2 = 1572 kN ç Governs

and $\alpha_1 \phi_c b_s t_c f'_c$ = (0.85 – 0.0015(25)) * (0.65) * 2.190m * 0.065m * 25,000kN/m^2 = 1879kN

We are connecting for 40% of this, so require a shear connection for 0.4 * 1572kN = 629kN

For 75mm deck with 19mm diameter x 115mm shear studs with a minimum edge distance of 65mm with w_d/h_d = 2.4 and 25MPa concrete, we get a factored shear resistance of 57kN/stud (see Table 5-3, shown earlier in Table 4.1).

Number of studs required each end of beam = 629kN / 57kN/stud = 11.03 studs. Say 11.

Approximate spacing = (8.75m) / (2 * 11 + 1) = 0.38m

We would require a minimum of 22 studs to save 8.75m x (59.5kg/m – 39.2kg/m) = 178kg. To determine if this is an economic solution, take the cost savings for the steel weight and divide by the number of studs required, determining the maximum cost per stud to make composite design economic. The steel cost should be taken as being the change in material only, as costs for detailing, fabrication and erection of the beam will be essentially unchanged.

At $1 / kg for change in steel weight = 178kg *$1/kg = $178 savings for this beam

However, this is partly offset by the extra concrete volume that is required for the wider deck flutes to surround the Nelson studs at say $150/m^3.

Extra concrete = 8.75m x 3m x (0.099m^3/m^2 – 0.087m^3/m^2) x $150/m^3 = +$47

Net saving = $178 – $47 = $131 / beam

With 22 studs, the saving is $131 / 22 studs = $5.95 / stud. As stud prices are often quoted at about $5 each, this would appear to be a reasonable, but not compelling, economic solution. An extra step in the construction process is required to install the studs after the deck is placed, but before the reinforcing steel is placed. The price of the studs will increase if there are only a small number of studs to be welded.

4.4 DEFLECTIONS UNDER COMPOSITE ACTION

Calculating the deflection of a composite beam is considerably more complex and involved than for that of a non-composite beam. S16 requires explicitly that the designer account for the effects of the increased flexibility arising from interfacial slip between the two materials, as well as creep and shrinkage of the

concrete. While all of these influences may be determined through tests or analysis, S16 provides a simple and direct approach in lieu.

The effect of slip between the steel and concrete components of a composite beam renders the composite stiffness a non-conservative estimate of the stiffness of the member. Even when a beam is designed for full (100%) composite action, there is a possibility of a small degree of interfacial slip that translates into increased beam deflection. S16 provides the following equation for estimating the effective moment of inertia, I_e, of the composite beam:

$$I_e = I_s + 0.85P^{0.25}(I_t - I_s)$$

where I_s is the moment of inertia of the steel section, p is the fraction of shear connection (1.0 for 100% shear connection), and I_t is the transformed moment of inertia of the composite beam (steel unit). As p increases, more of the composite stiffness is attributed to the transformed inertia, but the effective inertia never reaches the full transformed value. To account for concrete creep, S16 requires that any deflections that include the effects of long-term live loads, calculated using the effective moment of inertia, I_e, simply be increased by 15%.

With the concrete fully locked into the beam, shrinkage tends to impose curvature to the (unshored) composite member, pulling the centre of the beam downward and increasing deflection. (While the effect of concrete shrinkage is checked during the design of composite beams, this effect is not considered for non-composite beams as it is mitigated considerably by slip between the two materials.) Information on shrinkage of concrete in composite beams can be found in Annex H of S16, which forms the basis of the method in the Standard. The centreline shrinkage deflection, Δ_s, is calculated as:

$$\beta_s = \frac{L^2}{8} c \frac{\varepsilon_f A_c y}{n_s I_{es}}$$

where L is the composite beam span, A_c is the effective area of concrete slab (when corrugated steel deck is used, taken as the concrete above the flutes if the ribs are perpendicular to the span and the full concrete area if the ribs are parallel to the span), y is the distance between the centroids of the effective area of concrete slab and the (transformed) composite beam. The effective moment of inertia for determining shrinkage deflection, I_{es}, is analogous to I_e, except that the modular ratio, n_s, uses the age-adjusted modulus of elasticity of concrete, E'_c, when determining the transformed moment of inertia for shrinkage, I_{tr}. The age-adjusted modulus of elasticity of concrete is:

The remaining variables may be determined for specific design cases, but suggested values are also provided in Annex H. The free shrinkage strain of concrete, ε_f, may be taken as 583×10^{-6}, and the aging and creep coefficients of concrete, c and f, may be taken as 0.73 and 2.7, respectively. The parameter c is an empirical coefficient to account for concrete cracking, the non-linear stress–strain relationship of concrete, and other factors not captured by this simple method, and in the absence of project-specific research or more accurate data may be taken as 0.5.

While S16 permits the reduction of calculated deflections based on an assessment of the partial end fixity due to the continuous nature—even with simple connections—of composite construction, this effect is normally neglected unless deflections become a significant design issue.

The composite beam designed in the example in Section 4.3 is now assessed for deflections.

First, determine the deflection arising from interfacial slip and creep:

For serviceability limit states, unfactored live loads are normally used. In this case, one-half of the assembly live load will be taken as short term and the remainder as long term live load. This distinction affects the creep deflection calculation.

Unfactored live load = Unfactored area loading * Tributary width =
$= 4.8 kN/m^2 * 3.0m = 14.4 kN/m$

The values of I_s and I_t are conveniently found in the section data tables and the composite beams tables, respectively, in the *CISC Handbook*.

$$I_e = I_s + 0.85P^{0.25}(I_t - I_s) = 127 + 0.85(0.4)^{0.25}(486 - 127) = 370 \times 10^6\ mm^4$$

$$\Delta = \frac{5 * W_l * L^4}{384 * E * I} = \frac{5*(7.2+1.15*7.2)*8.75^4*1000}{384*200*370} = 16.0mm = L/548\ (OK)$$

Second, determine the deflection arising from concrete shrinkage:

Making use of the values recommended in Annex H for c, ε_f, c, and f, the determination of the shrinkage deflection is relatively simple. Moreover, the value of I_{tr}, using the recommended values for c and f, is tabulated in the composite beam tables in the *CISC Handbook*.

$$n_s = \frac{E}{E'_c} = \frac{200{,}000}{4500\sqrt{f'c}} = \frac{200{,}000}{4500\sqrt{25}/(1 + (0.73*2.7))} = 26.4$$

$$I_{es} = I_s + 0.85P^{0.25}(I_{tr} - I_s) = 127 + 0.85(0.4)^{0.25}(367 - 127) = 289 \times 10^6\ mm^4$$

$$A_c = 65mm * 2190mm = 142 \times 10^3\ mm^2$$

Let $\bar{y}$ be the distance from the bottom of the steel section to the neutral axis of the composite section. Taking the properties of the W410x39 section from the *CISC Handbook*:

$$\bar{y} = \frac{\Sigma Ay}{\Sigma A} = \frac{4990 * \left(\frac{399}{2}\right) + \frac{2190}{26.4} * 65 * (399 + 75 + \frac{65}{2})}{4990 + \frac{2190}{26.4} * 65} = 263mm < d = 399mm$$

$$\therefore y = 399 + 75 + \frac{65}{2} - 243mm$$

$$\beta_s = \frac{L^2}{8} c \frac{\varepsilon_f A_c y}{n_s I_{es}} = \frac{8750^2}{8} * 0.5 * \frac{583 \times 10^{-6} * 142 \times 10^3 * 243}{26.4 * 289 \times 10^6} = 12.6mm$$

Deflection calculations for composite beams need to be interpreted carefully. Camber is most often appropriate only to offset any dead load applied prior to the concrete curing (including the concrete itself). Specifying very small amounts of camber should be avoided due to the limited benefit in relationship to the associated cost, especially given that beams are typically erected with any natural camber in the upward direction. Live load deflections during construction are rarely an issue, but those of the composite beam when the building is in service must meet the serviceability limit state requirements. Because the long term deflections of the composite beam include dead load applied after the slab has cured, long term live load, and the effects of creep and shrinkage, it is good practice to evaluate this overall (quasi-permanent) deflection to ensure that the composite member is not excessively flexible, thereby inhibiting good performance. Assuming the beam is cambered for the non-composite permanent loading, using the equation for Δ under the long term live load plus the dead load of the partitions/finishes, and then adding Δ_s, gives a long term deflection of 22.3mm (L/392), which should be acceptable for most occupancies.

CHAPTER 5
AXIALLY LOADED MEMBERS

This chapter covers the background of axially loaded members. These can be members in tension or in compression. Members that are in compression will have their axial load capacity reduced dramatically as the unsupported length increases. This chapter describes the associated provisions in S16.

5.1 TENSION MEMBERS

The simplest type of member with axial load is the tension member as we are concerned only with yielding or tearing of the member. We are not concerned with stability.

Tension members are covered in S16 Clause 13.2 and there are three equations that govern. The tension resistance is the least of:

- Gross Section Requirement: $T_r = \emptyset A_g F_y$
- Block Shear Requirement: T_r = Determined by block shear equations of Clause 13.11
- Net Section Requirement: $T_r = \emptyset_u A_{ne} F_u$

where $\emptyset_u = 0.75$ and A_g and A_n are the gross and net areas and A_{ne} is the effective net area based on shear lag effects.

The first equation ($T_r = \emptyset A_g F_y$) applies to the middle of the member away from holes, notches or other disturbers of uniform stress. The second equation (block shear equations) applies at the end connections of a member and combines the resistance of both tension and shear. (We will review block shear failure when we get to connection design in Chapter 10.) The third equation ($T_r = \emptyset_u A_{ne} F_u$) applies at any location along the length of the member. For mild steel this governs at bolt holes or locations of reduced area and "A_{ne}" is the effective net area across the holes, as shown in Figure 5.1, and is determined from Clause 12.3.3.2 based on shear lag requirements.

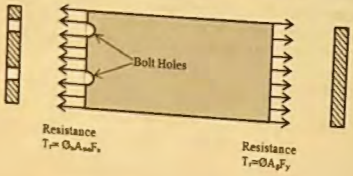


Figure 5.1. Stress distribution in tension member assumed for design

$A_{ne} = A_n$ when there is uniform stress.
$A_{ne} = 0.60A_n$ to $0.90A_n$ at bolted end conditions depending on the shape of the section and the number of holes. The most restrictive is $0.6A_n$ which is for angles connected by only one leg with less than four bolts in a longitudinal row.

Note that when we look across the bolt holes, only a short length is affected and we use F_u—the ultimate stress of the steel after strain hardening has taken place. For 300W or 350W in G40.21, this is taken as 450MPa. This variation is shown in Figure 5.2.

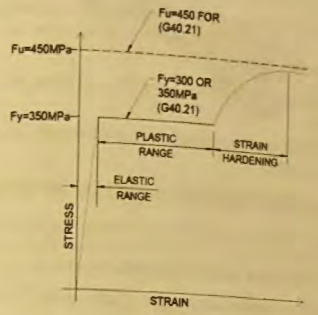
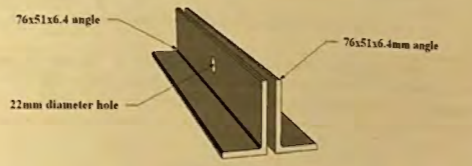


Figure 5.2. Idealized stress-strain curve for typical steel showing strain hardening

The block shear equations of Clause 13.11 take account of the efficiency of the end conditions and account for shear lag, which is an important issue to be accounted for at connections. (We will cover shear lag more extensively when we discuss connection design in Chapter 10.)

EXAMPLE—TENSION CAPACITY OF A BRACE WITH A HOLE

A brace that resists wind load and seismic loads in a "conventional construction" frame is formed from two L76x51x6.4 angles that are bolted together at intervals. Determine the factored tensile resistance Tr for this brace. The angles are G40.21 and have a yield Fy = 350MPa. There is a 22mm diameter hole through the longer edge of each angle for spacer-plate connections. (A 22mm hole would be appropriate for a 19mm bolt.) Ignore the connection design and related block shear and shear lag issues at this stage and consider just gross section requirement and net section requirements.



Solution:

Use both the gross and net sections and determine which is going to govern. This is the case of a hole in the middle of the member and the stress both "upstream" and "downstream" of the hole will be uniform. The net section therefore does not need to be reduced in the same way that is required for shear lag at the end of the member ($A_{ne} = A_n$).

Check tension resistance in the gross section using the angle properties found in Part 6 of the CISC *Handbook* the normal Ø of 0.9 and the yield stress.

Tension on gross section: $T_r = \emptyset A_g F_y = 0.9 * 2 * (766 \times 10^{-6} m^2) * 350{,}000 kN/m^2 = 483 kN$

Check the net section tension resistance using the net section area, the ultimate stress and a lower resistance factor, $\emptyset_u$ of 0.75, used to reflect that when using net section and ultimate stress there is less reserve strength to protect us against failures.

The hole in each of the two angles removes 6.4mm x 22mm = 141mm². (Use 22 + 2 = 24mm diameter if holes are punched.

Net section resistance: $T_r = \emptyset_u A_{nc} F_u = 0.75 * 2 * (766 - 141) \times 10^{-6} m^2 * 450{,}000 kN/m^2$

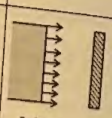
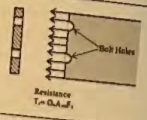
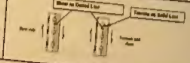
$T_r = 422 kN$

In this case the capacity is governed by the net section at the bolt hole and the brace is limited to 422kN.

5.2 SUMMARY OF TENSION MEMBER DESIGN

The design of tension members is summarized in Table 5.1, which gives an indication of what the usual governing loads are.

Table 5.1. Load the usually govern design of tension members

Failure Mode	At mid-span of member	At end connection of member	Figures
Gross Section $T_r = \emptyset A_g F_y$	Typically governs for mild steel	Typically won't govern.	
Net Section $T_r = \emptyset_u A_{nc} F_u$	May govern for high F_y (Shear lag effects do not apply)	Necessary to check this condition including effects of shear lag.	
Shear Block	Does not apply	Will need to do checks as this will usually govern capacity.	

5.3 PIN-CONNECTED TENSION MEMBERS

S16 also has special requirements for tension elements that are connected by pins through large shaped ends such as those shown in Figure 5.3, which have $T_r = 0.75 \emptyset A_n F_y$. The 0.75 factor comes in to account for the greater disruption of forces that occur around a large hole and the greater uncertainty in the distribution of forces around this hole. The designer should also check Clause 12.4 when looking at pin-connected members.



Figure 5.3. Typical pin-connected member

5.4 RESISTANCE OF MEMBERS WITH AXIAL COMPRESSION

Axial compression is present most often in columns and can occur with or without moments being present. We will start our discussion with members that do not have moment present.

The design equations for axial compression are given in S16 Clause 13.3.1. The requirements for Class 1, 2, 3 sections are as follows:

$$C_r = \emptyset A F_y (1 + \lambda^{2n})^{\frac{1}{n}} \text{ Where } \lambda = \frac{KL}{r}\sqrt{\frac{F_y}{\pi^2 E}} = \sqrt{\frac{F_y}{F_e}}$$

The value of n is 1.34 with two exceptions: (1) for Class H Hollow Structural Sections (which should not be used in building design due to difficulties in obtaining the material in a timely manner); and (2) for WWF sections made of plates with flame-cut edges (which are not used as axial elements except in tall steel structures or possibly in some unusual industrial structures).

Taking n = 1.34 for the majority of cases in building construction in our area we find:

$$C_r = \emptyset A F_y (1 + \lambda^{2*1.34})^{\frac{1}{1.34}} = \emptyset A F_y (1 + \lambda^{2.68})^{-0.746}$$

The formula above for λ **does not apply** to sections that are:

1) Class 4 (except with severe restrictions)
2) Singly-symmetric sections (e.g., tees, channels)
3) Asymmetric (e.g., angles – see discussion later in this chapter)
4) Cruciform sections
5) Sections that do not have a uniform cross-section throughout their length

For 345MPa steel λ will vary from 0 at no unsupported length to 2.66 at the highest permitted KL/r values of 200. While most engineers think in terms of KL/r ratios, the term λ (or λ^2) can be useful as it gives the relationship between the Euler Buckling stress F_e and the yield stress F_y by the equation shown as follows:

$$\lambda = \sqrt{\frac{F_y}{F_e}}$$

The formulas provided for axial capacity use λ to compute the axial resistance of the column, taking account of the theoretical buckling value and effects of mill tolerance, residual stresses, construction tolerances and the results of experimental test data. Accounting for these factors means that we do not just use the force in the member at theoretical buckling but reduce from that stress to get the stress permitted by S16.

Fortunately we do not have to work out this equation often as there are tables giving the value of C_r/A for various strengths of steel in Part 4 of the *CISC Handbook*. In addition, C_r, for various wide-flange and HSS shapes is given in the column capacity tables (green pages) of the *CISC Handbook*.

Through modification of the classical equation for the buckling of a column we can make a comparison to the code-permitted axial resistance of a column.

Start out with the classical equation for buckling: $C_r = \frac{\pi^2 EI}{(KL)^2}$

Substitute E = 200000MPa

Transform $r = \sqrt{\frac{I}{A}}$ to $r^2 = \frac{I}{A}$ and then to $I = A * r^2$ and substitute in the equation for C_r.

$C_r = \frac{\pi^2 * 200000 * (A * r^2)}{(KL)^2}$ transform and get $\frac{C_r}{A} = \frac{1974000}{\left(\frac{KL}{r}\right)^2}$

Using the formulas above it is possible to compare the theoretical buckling stress to the code resistance in terms of resistance axial stress C_r/A. The two equations for permitted resistance and theoretical buckling can be plotted for various values of KL/r. As shown in Figure 5.4, plotting these values clearly shows the loss of axial capacity as the unsupported length of the column increases. The axial load is restricted to allow for inaccuracies in the steel fabrication, built-in stresses due to fabrication and to provide a smooth transition from short to long columns. Note that S16 Clause 10.4.2.1 limits the maximum slenderness ratio to 200 for an element in compression. The C_r/A values are given in tabular form in Table 4-4 of the *CISC Handbook* for various F_y values at various KL/r values between 0 and 200.

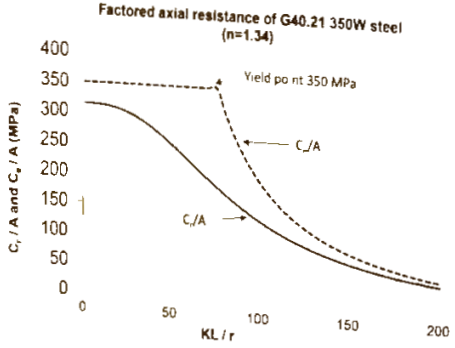


Figure 5.4. Axial resistance of 350W steel

EXAMPLE—CAPACITY OF PIN-CONNECTED COLUMN

Determine the axial resistance C_r for a pin-connected (K = 1.0) W200x59 350W column with an unsupported length of 5.0m; there are no moments present.

Solution 1:

The easiest solution to this problem is to look in the column capacity tables (green pages) of the *CISC Handbook* and find that for a W200x59 column with a KL of 5000mm the C_r is 1060kN. (Similar to the beam selection tables (blue pages) of the *CISC Handbook* that set out tables we have been using for beam design, the column capacity tables (green pages) of the *CISC Handbook* use a yield stress of 345MPa in their calculations.)

Solution 2:

The second easiest solution is to determine the minimum radius of gyration from the wide-flange shape tables in Part 6 using the properties for a W200x59 $r_x = 89.9mm$ $r_y = 52.1mm$, $A = 7530mm^2$

KL/r = 1.0*5000mm / 52.1mm = 95.7 say 96 (note the minimum value of r used)

From table, $C_r/A = 141MPa$

$C_r = 141{,}000kN/m^2 * 7530 \times 10^{-6}m^2 = 1062kN$

Solution 3:

The third solution is to work out the equations given in the standard:

$$\lambda = \frac{KL}{r}\sqrt{\frac{F_y}{\pi^2 E}} = \frac{1.0 * 5m}{0.052m}\sqrt{\frac{350MPa}{\pi^2 * 200000MPa}} = 1.2804$$

$$C_r = \phi A F_y (1 + \lambda^{2n})^{\frac{1}{n}}$$

$$C_r = 0.9 * 7530 \times 10^{-6}m^2 * 350{,}000\ kN/m^2 * (1 + 1.2804^{2.68})^{\frac{1}{1.34}}$$

$$C_r = 2372kN * (2.940)^{-0.746} = 2372kN * (0.4472) = 1061kN$$

5.5 B-OVER-T RATIOS FOR SQUARE AND RECTANGULAR HSS SECTIONS

To determine the section class of rectangular and square hollow structural sections (HSS), we must determine their b-over-t (b/t) ratio. As the b/t ratio will frequently be of interest when doing seismic design, it is desirable to have a common basis to evaluate the ratio.

Clause 11.3.2(b) allows us to compute the b/t ratios for rectangular and square HSS sections using the "nominal outside dimension less four times the wall thickness." The dimension of the nominal outside dimension, less four times the wall thickness, is equal to the flat length of the side of the HSS. The radius of the corners of HSS member was established in the 1973 Stelco Manual on HSS connections, which provides the definition for the radius as shown in Figure 5.5.

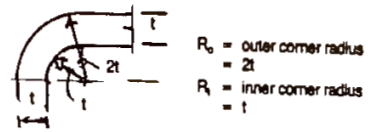


Figure 5.5. Definition for the radius
Source: Stelco 1973 manual on HSS Connections

Packer and Henderson *Hollow Structural Section Connection and Trusses* (1973) further amplify this as follows:

> The outside radius of rectangular HSS corners is generally taken as two times the wall thickness for design considerations and for the calculation of physical properties. Production corner radii may vary from this within the CAN/CSA-G40.20-M92 tolerances shown in Table 1.3. The actual outside corner radius can become a practical consideration when details of joint preparation for welding procedures are being developed for connections.

(Note that Table 1.3 as referenced by Packer and Henderson is identical to that shown in Part 6 of the *CISC Handbook*.)

Table 5.1 shows b/t ratios for rectangular HSS sections (in G40.21) that employs the definition of outer corner radius being twice the wall thickness. Table 5.2 shows b/t ratios for round HSS sections (in G40.21) that employ the definition of corner radius being twice the wall thickness. In keeping with the practice of Packer and Henderson, the actual wall thickness rather than the rounded dimension is used for calling up the section on drawings.

Table 5.1. b-over-t ratios for rectangular HSS sections (G40.21)

b is taken as outside dimension minus 4 times wall thickness												
<---------- Outside Dimension of the HSS ---------->												
Wall	**38**	**51**	**64**	**76**	**89**	**102**	**127**	**152**	**178**	**203**	**254**	**305**
3.18	7.9	12.0	16.1	19.9	**24.0**	**28.1**						
3.81	6.0	9.4	12.8	15.9	19.4	22.8						
4.78	3.9	6.7	9.4	11.9	14.6	17.3	**22.6**	**27.8**	**33.2**			
6.35		4.0	6.1	8.0	10.0	12.1	16.0	19.9	**24.0**	**28.0**	36.0	44.0
7.95				5.6	7.2	8.8	12.0	15.1	18.4	21.5	**27.9**	**34.4**
9.53				4.0	5.3	6.7	9.3	11.9	14.7	17.3	**22.7**	**28.0**
11.13							7.4	9.7	12.0	14.2	18.8	**23.4**
12.70							6.0	8.0	10.0	12.0	16.0	20.0

Class 1 = $\frac{b}{t} \le \frac{420}{\sqrt{F_y}} = 22.45$

Class 2 = $\frac{b}{t} \le \frac{525}{\sqrt{F_y}} = 28.06$

Class 3 = $\frac{b}{t} \le \frac{670}{\sqrt{F_y}} = 35.81$

Class 4 = Assumes F_y = 350MPa and not ASTM A500 sections)

Table 5.2. b-over-t ratios for round HSS sections (G40.21)

Wall thickness / outside diameter																	
<----------Outside Dimension of the HSS---------->																	
Wall	**48**	**60**	**64**	**73**	**76**	**89**	**102**	**114**	**127**	**141**	**152**	**168**	**178**	**219**	**273**	**324**	**406**
3.2	15.0	18.8	20.0	22.8	.	27.8	31.9	35.6	**39.7**		**47.5**	**52.5**					
4.8	10.0	12.5	13.3	15.2	15.8	18.5	21.3	23.8	26.5	29.4	31.7	35.0	37.1	**45.6**	56.9		
6.4		9.4	10.0	11.4	11.9	13.9	15.9		19.8	22.0	23.8	26.3	27.8	34.2	**42.7**	**50.6**	63.4
8.0							12.8		15.9		19.0	21.0	22.3				
9.5								12.0	13.4	14.8	16.0	17.7	18.7	23.1		34.1	**42.7**
13.0									9.8			12.9	13.7	16.8	21.0	24.9	31.2

Bending only:

Class 1 = $\frac{b}{t} \le \frac{13000}{F_y} = 37.1$

Class 2 = $\frac{b}{t} \le \frac{18000}{F_y} = 51.42$

Class 3 = $\frac{b}{t} \le \frac{66000}{F_y} = 188.5$

Class 4 (Assumes F_y=350 MPa and not ASTM sections)

What we see from both Tables 5.1 and Table 5.2 is that there are some Class 3 and 4 bending sections, especially in the larger square and rectangular sections with the thinner walls. For axial capacity, the section limits are derived from S16 Table 1. Which gives the following limits:

Square and Rectangular: $\frac{b_{el}}{t} \le \frac{670}{\sqrt{F_y}} = 35.8$ for F_y=350 Round: $\frac{D}{t} \le \frac{23000}{F_y} = 65.7$ for F_y=350

Most commonly available round HSS sections meet Table 1 requirements. For square and rectangular sections not meeting table 1 conversion of area used in axial calculations can be done by reducing the theoretical size of the HSS to the point where it meets the Table 1 requirements as shown in figure 5.6 you then use A_e with Clause 13.3.5. The inertia used in the calculation of C_e is the inertia of the 305x305 prior to reduction to A_e.

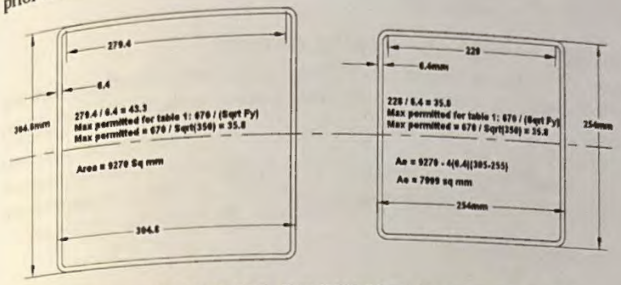


Figure 5.6. Calculation of A_e for HSS not meeting Table 1 requirements

5.6 AXIAL RESISTANCE OF SINGLE ANGLE BRACES

The single angle brace is usually not considered as being covered under the conventional compression resistance formula:

$$C_r = \phi A F_y (1 + \lambda^{2n})^{-\frac{1}{n}}$$

This is for three reasons:

1) the section is not symmetric (singly-symmetric if the legs are equal);
2) single angles braces are usually loaded eccentrically, typically by bolts or welds to one leg; and
3) the angle section has a principal axis that is not parallel to either of the legs of the angle, as shown in Figure 5.7.

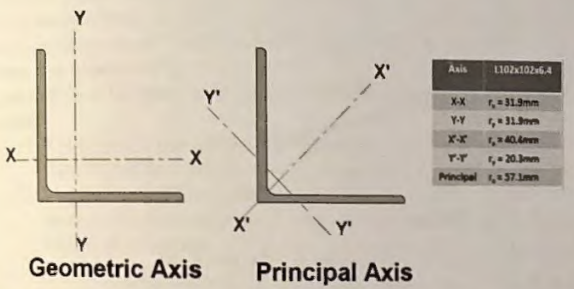


Figure 5.7. L102x102x6.4 showing traditional axes (left) and principal axes (right)

S16-09 has brought in a special clause on the axial capacity of single angle braces which depends in part on their end connection. The new formulas work by determining a KL/r that is appropriate and takes account of the loading conditions at the end of the member. Examining S16 Clause 13.3.3 we can find the axial resistance of an angle brace with no transverse loads that is connected by two bolts at each end with only one leg loaded (such as in Figure 5.8).



Figure 5.8. Angle brace with no transverse loads that is connected by two bolts

EXAMPLE—COMPRESSION CAPACITY OF AN ANGLE BRACE

Determine the axial capacity of a 2m long single angle 102x102x6.4 brace that is connected at each end to a gusset plate by 2 bolts (the same leg of the angle receives the bolts on each end and there are no transverse loads). The angle is 300W G40.21.

Using the procedure of Clause 13.3.3.2:

Useful section properties: $r_x = 40.4$mm, Area = 1260mm^2

Use formula (a) from S16 Clause 13.3.3.2 as L/r_x is less than 80.

$$\frac{KL}{r} = 72 + 0.75 * \frac{L}{r_x} = 72 + 0.75\left(\frac{2000}{40.4}\right) = 109$$

We could use the formulas given in 13.3.3.2 but it is easier to realize these formulas are the same as the standard formulation for C_r/A and standard tables can be used. We can now go to Table 4-4 of the *CISC Handbook* and find using $F_y = 300$ that $C_r/A = 113$MPa.

$$C_r = 113{,}000\text{kN/m}^2 * 1260 \times 10^{-6}\text{m}^2 = 142\text{kN}$$

Extending this to multiple lengths we can produce the capacity graph shown in Figure 5.9.

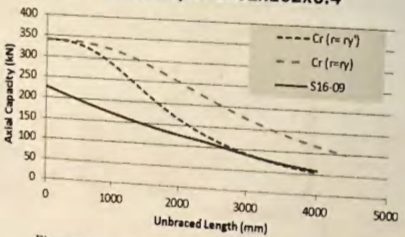


Figure 5.9. Single angle braces loaded by bolts in one leg
(Note: Clause 13.3.3.2 does not apply to long braces or brace bays)

What we see in Figure 5.9 is that single angle braces loaded by bolts in one leg only have considerably less capacity than we might have considered if we had considered the angle as uniformly loaded and used even the minimum radius of gyration.

Additional topics pertaining to column design can be found in Chapter 9.

CHAPTER 6
TRUSSES AND OPEN WEB STEEL JOISTS

Elegant exposed steel structures can be constructed using hollow structural section (HSS) trusses in either two-dimensional form as plane trusses or three-dimensional format as space trusses. Exposed HSS trusses are usually an interaction between the architect with his or her requirements for aesthetics and the structural engineer with his or her requirements for strength and stability. Throw in the added constraint of cost, and this can pose many challenges to finding a solution that satisfies all criteria.

6.1 HSS TRUSS DESIGN

While many designers may consider the main task in designing HSS trusses to be determining the forces and sizing the members, the real art is in sizing the members so that the connections will work without extra charges at the shop drawing stage.

All HSS members should be Class C. While Class H sections are listed in the *CISC Handbook*, these sections are hard to source and it is difficult to determine in the field if the higher-grade class of section has been supplied. HSS sections should satisfy CAN/CSA-G40.20 in 350W and this requirement should be stated on the structural drawings. The use of ASTM sections should be avoided in trusses designed under S16 unless the wall thickness has been increased to take account of the manufacturing tolerances of the ASTM material.

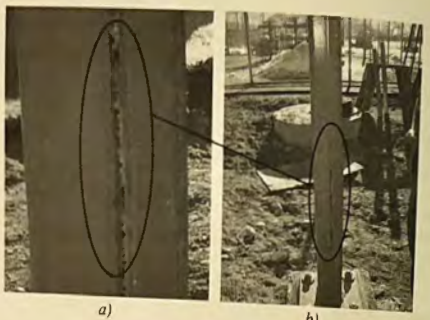
ASTM A500 specification permits the walls of HSS members to be up to 10% below the stated wall thickness, while G40.20 sections are only permitted to have wall thicknesses equal to or greater than the stated wall thickness. Reducing the wall thickness will have a significant impact on the ability of the section to resist connection forces, and ASTM A500 steel should only be accepted as a substitute for G40.20 after a re-evaluation of the member strength and joint resistance using members that have a wall thickness equal to 90% of the stated wall thickness. ASTM A500 sections should be avoided for the following two reasons:

1) The thinner walls will have a significant impact on the capacity of the truss joints – information from Packer and Henderson is based on empirical testing using G40.21 steel.
2) The axial capacity of similar-sized ASTM sections will be lower than the equivalent G40.21 sections due to tolerance and lower yields. It may be desirable when producing drawings to give a table of acceptable ASTM alternatives that clearly shows that the wall thickness increases if ASTM material is to be used.

The current edition of the *CISC Handbook* should be used when designing HSS trusses and selecting member sizes as several sizes of HSS section are listed in the 7th edition of the *CISC Handbook* that are not listed in the 8th or 9th edition. Sourcing of sections not listed in the current handbook may be difficult and they should not be called up.

The interaction between aesthetics and structure will also influence the selection of section profile. The architect may wish that a certain profile of tube be used (round, square or rectangular) and will most certainly desire a uniformity to the member sizes selected. Choosing member sizes in an architecturally exposed truss based solely on the strength requirements will seldom occur.

All HSS members should be seal welded to prevent water getting in that can cause problems with internal corrosion and splitting of the pipe during freezing weather. Even if the tube is to be in a warm environment after the completion of the building, splitting during freezing weather can occur during erection if water is able to get into the tube. This is shown in Figure 6.1 where an HSS column used to support an exterior stair has split from freezing water.



a) b)

Figure 6.1. a) Effect of water freezing inside an HSS section. b) The freezing water has split the HSS and replacement with a sound member that was free of water was required

Under S16, a simplified method of truss design was introduced in 2001 that considers only the axial forces in the truss members along with the chord primary bending moments due to loading of the chord between the nodes. Trusses designed using this procedure may be designed without having to consider the secondary moments that come from truss deformation with continuous chords and welded web members provided:

1) The out-of-plane stiffness of all compression members is not less than the in-plane stiffness.
2) The compression members are Class 1, 2, or 3 (Class 4 is not permitted).
3) There are no abrupt changes in the slope of the chords.
4) The truss is not a Vierendeel truss.
5) The truss is not bottom-chord bearing.
6) The truss is not subject to fatigue loading.

The simplified method recognizes that while there will be some secondary moments in trusses satisfying these criteria, treating the truss as a pin-connected truss produces a reasonable design provided there can be some redistribution of the secondary moments in the joints.

S16 has the requirement that the bottom chords of trusses be braced to give a maximum KL/r value of 240. The Commentary states that this is to facilitate handling during erection, but in the Standard it would appear that this is required to limit sag, vibration and slack under service conditions. It should be kept in mind that if the truss is subject to uplift or other force reversals that would put compression in the bottom chord, S16 would require the KL/r be limited to 200 like any other compression member.

Similar to the requirement on the first compression web members in joists, S16 Clause 15.2.5 requires that "The factored resistance of the first compression web member subject to transverse shear, and its connections, shall be determined with their respective resistance factors, Ø, multiplied by 0.85." When doing automated design using computer programs, you should verify that the program has correctly identified

which is the first compression diagonal and is applying this rule. When designing by hand or checking a computer design by hand, it would be easier to increase the loads in the first compression diagonal by 1/0.85 = 1.18 and use the normal design charts.

6.2 HSS TRUSS MEMBER SIZING FOR STRENGTH

Member sizing of HSS trusses is performed using the following equations:

$$\frac{T_f}{T_r} + \frac{M_f}{M_r} \le 1.0 \quad \text{for tension members}$$

$$\frac{C_f}{C_r} + \frac{U_1 M_f}{M_r} \le 1.0 \quad \text{for compression members. (Note: no 0.85 factor, as HSS are not I-shaped)}$$

For most cases except where there is a large loading on the compression chord member between panel points, the U_1 factor can be conservatively estimated to be 1.0.

While it might be possible to assume a K value less than unity for chord members, this is often overlooked in the interests of reviewing the large number of members inherent in many HSS trusses. Especially if the member is supported against buckling in any direction at the end nodes, the unsupported length for compression calculations will often be taken as the distance between node centrelines. When sizing by hand it is usual to use the column capacity tables (green pages) of the *CISC Handbook*.

EXAMPLE—COMBINED AXIAL AND BENDING ON AN HSS MEMBER

Following the frame analysis of an HSS truss web member it is found that under factored dead and snow load with wind effects added the member has a tension of 210kN and a moment of 6kNm, while under factored net wind uplift the member has a compression load of 85kN and a moment of 3kNm. The length of the member measured between the centreline of nodes is 3.5m. Determine the minimum size member from a strength standpoint. Member is to be round and G40.21.

Solution:

On the basis of carrying 85kN over 3.5m try 89 Diameter x 4.8mm.

From the table in Part 4, $C_r = 133kN$, $M_r = 10.7kNm$, $T_r = 397kN$

Use L = 3.5m Use K = 0.75KL = 2.6m

$$C_e = \frac{\pi^2 * E * I}{(KL)^2} = \frac{\pi^2 * 200 * 10^6 kN/m^2 * 1.12x10^{-6} m^4}{(0.75 * 3.5m)^2} = 3201kN$$

$$U_1 = \frac{1}{1 - \frac{C_f}{C_e}} = \frac{1}{1 - \frac{85kN}{3201kN}} = 1.36$$

$$\frac{C_f}{C_r} + \frac{U_1 M_f}{M_r} = \frac{85kN}{133kN} + \frac{1.36 * 3kNm}{10.7kNm} = 0.639 + 0.380 = 1.02 \text{ Almost OK}$$

$$\frac{T_f}{T_r} + \frac{M_f}{M_r} = \frac{210kN}{397kN} + \frac{6kNm}{10.7kNm} = 0.530 + 0.561 = 1.09 > 1.0 \text{ No go}$$

Increase to HSS 89 Diameter x 6.4 at 3.5m $C_r = 170kN$, $T_r = 520kNm$ $M_r = 13.7kNm$

$$\frac{T_f}{T_r} + \frac{M_f}{M_r} = \frac{210kN}{520kN} + \frac{6kNm}{13.7kNm} = 0.404 + 0.438 = 0.84 < 1.0 \text{ OK}$$

A check of the compression equation will show that this also works.

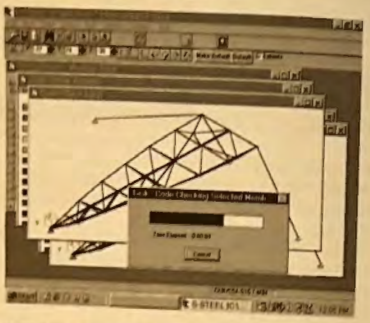
Result:
Use HSS 89 Diameter x 6.4 (G40.21)

Note: Given the difficulty in obtaining round HSS in G40.21 it would be appropriate to use ASTM A500 for this check and calling up ASTM A500 round sections. Care must be taken in looking at joint capacities with ASTM A500 as Packer and Henderson base their results on G40.21 steel and wall thicknesses.

6.3 AUTOMATED DESIGN OF HSS TRUSS STRUCTURES

HSS truss structures can often have hundreds if not thousands of members. In addition, there may be several load combinations to evaluate with partial and unbalanced snow loading, wind uplift, etc. This very quickly results in a substantial problem, not from an engineering standpoint, but from the standpoint of keeping track of the results and working through all the necessary analysis. For this reason, it is common to automate this process using programs such as Softek's S-Frame and S-Steel, which can very quickly analyze the interaction ratios for all the members in the truss and print out in graphical or tabular form the interaction ratios. Sample outputs are shown in Figure 6.2.

a)



b)

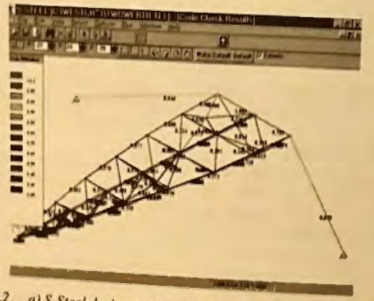


Figure 6.2. a) S-Steel during code check of the member. b) Results from an S-Steel run where the values shown are the interaction ratios. It is also possible to have the program select member sizes but this may lead to results that are not architecturally acceptable

Where possible, the design of HSS trusses should be simplified by reducing the choice of sections used. It is usual to rationalize the truss design and have a single member size and thickness in common locations. For example, if a truss needs a HSS102x102x6.4 web member at the ends for strength, it would be common to use this member for all web members on that truss even if the web members toward the centre of the truss could be reduced to 102x102x4.8 or 89x89x4.8. Caution should therefore be taken when allowing automated structural design packages to choose member sizes to verify that common-sense rules of sizing the member are not overruled. Caution should also be taken when using automated structural design programs as they will not perform the calculations necessary to verify if the joints are acceptable. Verifying the joints are acceptable is a fundamental part of HSS truss design. Automated steel design programs may also not review the deflection requirements of individual members or of the truss as a whole. For these reasons it is not uncommon to have many of the interaction ratios produced by a code check run to be less than 0.7.

6.4 SELECTION OF RECTANGULAR OR ROUND SECTIONS

HSS members come in square, rectangular, and round shapes. The architect, for aesthetic reasons, will often select the member "alphabet." However if there is a choice the following points should be kept in mind:

1) Round sections are generally easier to form connections if the truss is three dimensional in shape. This avoids "bird-mouthing" where the end of a rectangular section meets two faces of another rectangular section.
2) Square or rectangular sections work well if the truss is in a single plane or if the truss is formed from a group of single plane trusses.
3) Attachment of deck to round members requires that a channel be attached to the round section to provide a flat place for the deck to be supported.

6.5 JOINTS FOR HSS TRUSSES

Joints for HSS trusses are formed by one of the following alternatives:

1) By "profile cutting" the smaller web member so that it fits snugly to the longer chord members and is then welded all around. This is the most common method of making HSS trusses, especially those that are architecturally exposed.
2) By flattening the ends of the web member, forming a flat surface that can be welded to chord members or bolted to plates that are welded to the chord members.
3) By using plates that are slotted into the HSS web members and then bolted to plates that are welded to the chord members.
4) By using patented connectors that are welded to the ends of the members. A sample of this type of truss connector in use is the (now former) space frame over the Plaza of Nations at the B.C. Pavilion.

6.6 HSS CONNECTIONS

HSS connection design is based on experimental work covering a limited range and type of joints. The designer is well advised to stay within these limits if extra charges are to be avoided. The definitive work on HSS connection design for Canada is *Hollow Structural Section Connections and Trusses: A Design Guide* by Packer and Henderson. A copy of this book is essential for HSS truss design. Some of Packer and Henderson's tables with joint capacities have been reproduced in the Ipsco catalogue and manual.

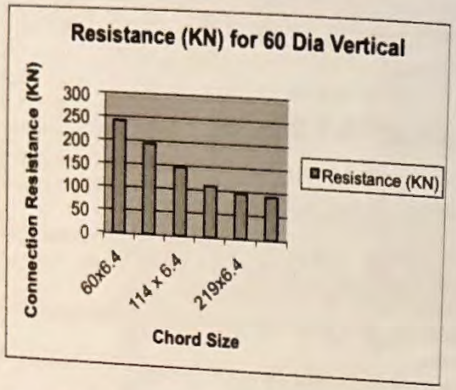
The capacity of joints is influenced by many factors, including:

- Geometry of joint
- Member profile (Square, rectangular, or round)

- Member size
- Relative size of chords and diagonals
- Chord wall thickness
- Whether the truss is 2D or 3D

The capacity of the joint is reduced if there is an axial compression present (i.e., at the compression chord). The formula for this can be found in the Ipsco manual as well as in Packer and Henderson.

The capacity of a joint to resist load decreases radically if there is a large discrepancy in the size of the connected members. This is illustrated in the graph in Figure 6.3.



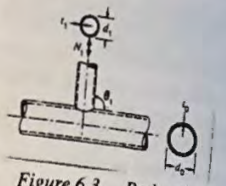
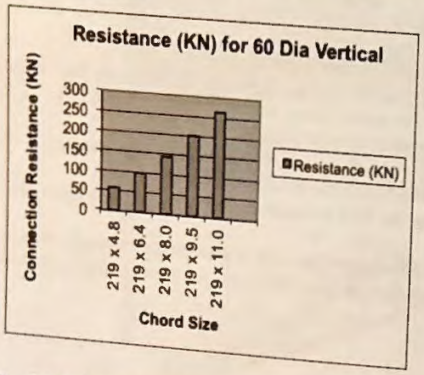


Figure 6.3. Reduction in capacity of joint to resist load as the size of the chord increases. (It's easier to punch a small web into a large chord than it is to punch the web into the chord if they are equal size.)

The capacity of the joint increases as the wall thickness of the chord member increases, as illustrated in the graph in Figure 6.4.



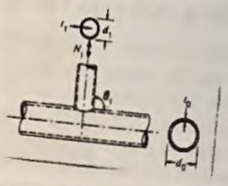


Figure 6.4. Increase in capacity of joint to resist load as the wall thickness of the chord increases. (It's easier to punch a small web into a chord with a thin wall than it is to punch the web into the chord with a thick wall)

6.7 GAP AND OVERLAP CONNECTIONS

For trusses where two web members intersect at the chord, there is a choice between a gap or an overlap connection. The choice of connection will depend partly on the geometry of the connection and the desire on the part of the designer to reduce the eccentricity of the joint. Gap and overlap connections are shown in Figure 6.5.

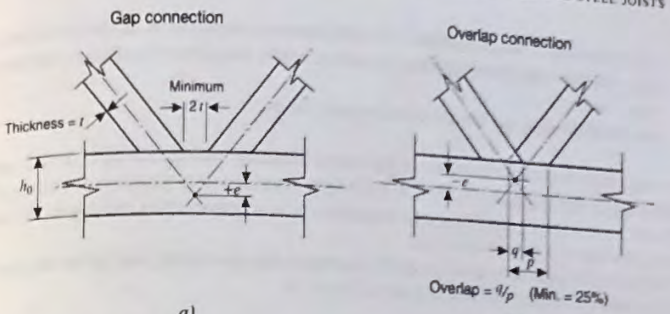


Figure 6.5. a) Gap connection. b) Overlap connection
Source: Former Ipsco catalogue

6.8 GENERAL RULES FOR HSS TRUSSES

The following information is taken from Packer and Henderson and provides some general rules of thumb and other suggestions to make the conceptual design of HSS trusses easier:

- Chords should have thick walls to assist with connection design.
- If you have a choice of two possible chord members (say between a 168 diameter x 8.0 and a 219 diameter x 4.8), it is better to choose the smaller diameter with a thicker wall.
- Web members will be more efficient if a larger diameter is chosen with a thinner wall.
- Web members should be Class 1 or 2 for gap connections.
- Web members should be Class 1 for overlap connections.
- For rectangular chord and web members, make chord members larger than the diagonals to assist in connection welding. For round chord and diagonal members, the chords should be the same size or larger than the diagonal members.
- Gap connections are preferred. When fitting members together, it is only necessary to fit each member over one member in a gap connection. In contrast, overlap connections require members to be placed over two members.
- If using overlap, at least one-quarter of the overlapped member must be overlapped.
- Do not have members intersecting at less than 30 degrees as it makes welding difficult.
- Use of deeper trusses reduces force on chords, but the deeper trusses have longer web members and require bigger web members to satisfy computed truss forces.
- For HSS trusses, the most economical span-to-depth ratio is between 10 and 15.
- As the deflection of trusses is a combination of bending deflection and shear deflection due to stretching of web members we can account for the shear deflection when doing hand calculations by using moment of inertia from chords alone and reducing the moment of inertia by 15% to account for shear deflections.
- Minimum fillet weld for web members should be 1.10 times the wall thickness of the web member.
- Finally, don't forget to draw out some joints to examine the configuration.

6.9 EFFECTIVE LENGTH (KL) OF HSS TRUSS MEMBERS

For HSS trusses constructed from profile-cut members, there will be some fixity due to the welding at the ends of the web members and the continuity of the chords that will improve resistance. Packer and Henderson recommend that, for welded HSS trusses, the length of the members should be multiplied by K as follows:

- For web members K = 0.75, where L = length between centreline of chord and centreline of chord, note that due to the size of the chord member L will be longer than the distance between face of chord members.
- For chord members use K = 0.9 between panel points.

Various rules apply regarding the amount of gap and the amount of overlap. For more details, see Packer and Henderson. Some sample joints must be drawn up to determine what kind of joint will result.

It is desirable to specify what the joint configuration is, as is shown in the following detail taken from structural drawings of a walkthrough truss.

It is necessary to give some guidance on the desired configuration of the joints in the structural drawings for the detailer. Such an example is given in Figure 6.6.

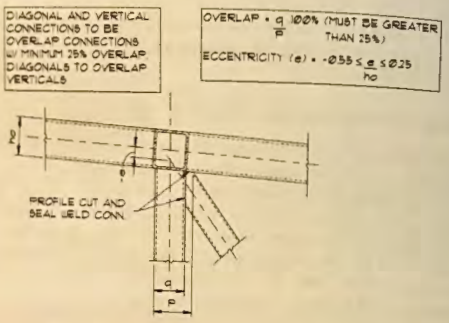


Figure 6.6. Sample detail from structural drawings showing the desired rules for truss configuration

6.10 EXPRESSING HSS TRUSS MEMBER FORCES ON DRAWINGS

As the connections for HSS trusses will usually be designed at least in part by the fabricator's engineer and detailer it is necessary to include in the structural drawings the forces. This can be done by providing a range of tension and compression forces for each type of member.

6.11 COMPUTER MODELLING OF HSS TRUSSES

Computer modelling of HSS trusses involves two issues:

1) Should the members, especially the web members, be considered pin- or moment-connected at their ends? Packer and Henderson recommended that if the length of the web members is at least 6 times the diameter of the web member in the plane of the truss, then the web members can be modelled as pin members.
2) Should the eccentricity of the joint be modelled? The answer to this question depends on the degree of eccentricity. Joints where the members are all concentric obviously need no special modelling, but if the joint is such that a significant eccentricity occurs, it will be necessary to have small rigid members coming from the centre of the member modelling the chord to the point where the web members intersect. Additional guidance is provided in Packer and Henderson.

These issues are illustrated in Figure 6.7.

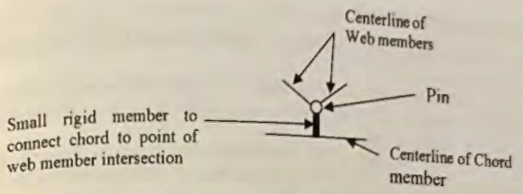


Figure 6.7. Computer modelling of joint eccentricity for truss

6.12 STABILITY OF THE TOP CHORD OF OPEN OR THROUGH TRUSSES

Many trusses will be located such that the top flange is attached to a stabilizing diaphragm. In some cases this arrangement does not work – there will not be enough room below the diaphragm and there will be need a handrail that can be provided by locating the compression chord of the truss higher. An example of such systems is the gangway to most docks. As the top chord is not supported by a diaphragm, care must be taken to make certain that the top (compression) flange is prevented from buckling laterally.

A similar problem to the top chord stability issue is the walkthrough (or "pony") truss. In this case, the top chord of the truss is supported by a diaphragm but often the only way that the diaphragm is stabilized is by out of plane bending of the diagonals of the truss or a portal frame is used at each end of the truss. The truss is positioned such that the bottom chords support the walkway deck and the upper chords act as a guardrail and handrail. The problem with this structural form is that the compression chord is not directly supported. Support must be provided to prevent lateral buckling of the compression chord and to carry the guardrail or wind forces down to the walkway, which will act as a diaphragm usually by being trussed in plan. The usual method of providing this support is to call up a moment connection where the verticals meet the floor members such that they can cantilever up to the compression chord. The minimum lateral force should be 2% of the chord force with factored guardrail forces added in where appropriate. Where moment connections are required for stability they should be called up on drawings such that they will be included in the joint design prepared by the detailer. Shown in Figure 6.8 is a truss where the top chord is supported against lateral buckling by the verticals at the side of the truss being moment-connected to the bottom members of the truss.

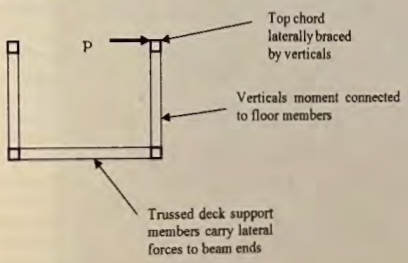


Figure 6.8. Trough truss where lateral restraint of the top flange is provided by the verticals that are moment connected to the floor members

6.13 WIDE-FLANGE TRUSSES

Many trusses that carry heavy loads or span long distances are constructed from wide-flange sections. There are several reasons for this:

- The area available in the largest size of square or rectangular HSS section is about 14,200mm², while W360 members used for compression can go to over 80,000mm², meaning that the higher forces can be accommodated.
- The wide-flange sections have a much higher bending capacity than the HSS sections even when bending about the weak axis, and this helps if the top chord of the truss is trying to pick up loads between the truss panel points.
- The connections between wide-flange elements can be done easily, especially if the web of the chord members is horizontal.

There are two options for the orientation of the webs of the wide-flange trusses. The first of these is to have the webs vertical as shown in Figure 6.9a and b. This will generally require stiffeners where the chords come in and the connections will involve a lot of welding.

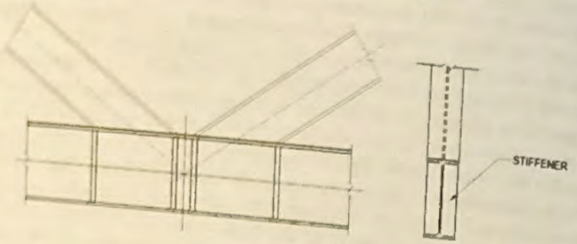


Figure 6.9a. Option 1 for wide-flange trusses



Figure 6.9b. Wide-flange trusses using web of wide-flange members vertical. The stiffeners required at the connections can clearly be seen.

The second option is to have the webs horizontal as shown in Figure 6.10. This results in much easier connections as the chords and diagonals can be connected by gusset plates that are bolted in place.

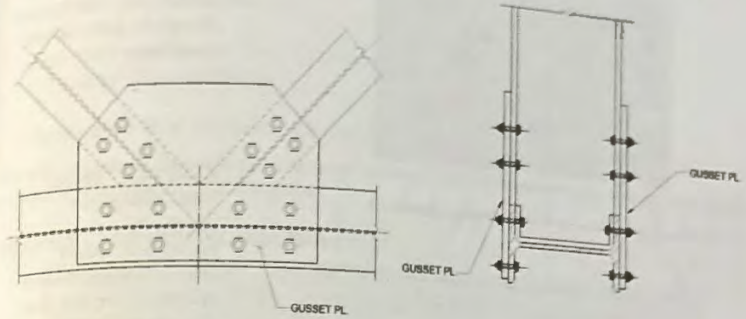


Figure 6.10. A second option for wide-flange truss configuration. In this option the webs are horizontal

For the second option with the web horizontal, some engineers will feel that the sections will be too weak and not resist bending effectively. However, for W360 column sections the bending about the "weak" axis can have a resistance of over 1,000kNm. Provisions must be made at the top chord to support joists or deck as the width of the flange alone is too narrow. Very heavy chord members will have a member depth (d) greater than their nominal size (e.g., a W360x196 has a depth of 372mm) and shims may be required to be attached to the diagonals members to make the gusset plates work. Most fabricators will prefer Option 1, where the webs are horizontal.

6.14 OPEN WEB STEEL JOISTS

Open web steel joists (OWSJ) are an economical alternative to rolled sections when designing floors and roofs. This is illustrated by the frequency of their use in tilt-up or other "big-box" type stores.

From the building designer's standpoint, OWSJs are essentially a design-build item, with the building engineer setting the parameters to be used by the joist fabricator and his or her engineer who does the detail design. The engineer of record for the building determines the depth and layout of the joists and provides structural drawings showing this layout along with loading information. The joist engineer does the detail design on the joists and prepares sealed calculations and shop drawings. The engineer of record then reviews the shop drawings for the joists to make sure that the configuration and input loading are correct.

The building engineer will lay out the joists, keeping in mind the column layout, building geometry and deck capacity. The selection of the depth of the joist is also carried out by the building structural engineer and is done using catalogues provided by the fabricator. As the detailed design is prepared by the joist fabricator, it is necessary that the structural drawings accurately and fully describe the geometry and desired loading on the joist.

While it is possible to fire rate OWSJs, this is not as easy to do as the fireproofing of rolled steel beams. OWSJs will have a greater cost advantage over steel beams where fireproofing is not required.

Because the discrete diagonal webs of OWSJs are much more stressed than the continuous plate webs of rolled steel beams, shear deflections on OWSJs are more critical than they are with rolled sections. An installed OWSJ is shown in the photograph in Figure 6.11 and in the section drawing shown in Figure 6.12.



Figure 6.11. Installed OWSJ. The deck sits directly on the double-angle top chord. Ducts and sprinkler piping can be seen passing through the joists without interference

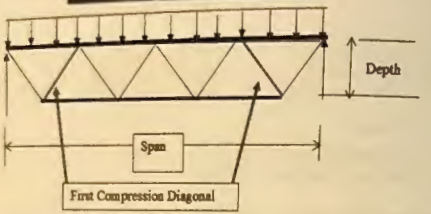


Figure 6.12. Typical OWSJ arrangement showing the typical loading leading to combined compression and bending in the top chord. The first compression diagonal each end is highlighted

EXAMPLE—JOIST SIZING

Using the data in Table 6.1, select the size of an OWSJ located in the roof of a warehouse building located in an area where $S_s = 1.5$ and $S_r = 0.2$. Joist span is 8.0m and tributary width is 2.0m. Downward wind, including interior and exterior components (but not including load factors), is 0.38kPa. Deck is 38mm with a 0.8kPa allowance for roof loading and 0.2kPa for suspended sprinkler / duct / lighting loads. Roof is essentially flat. The building is 40m x 50m and there are no obstructions to cause snow drifting, and ponding is not an issue. There are no architectural restrictions as to beam depth.

As maximum roof dimension is less than 70m the C_b for flat roof is 0.8.

Snow loading = $(C_a = 1.0) * (C_b = 0.8) * (C_w = 1.0) * (C_s = 1.0) * (S_s = 1.5) + (S_r = 0.2) = 1.40$kPa

Table 6.1. Roof loading in Richmond

ROOF LOADING IN RICHMOND				
Load Source	**Dead (kPa)**	**Snow (kPa)**	**Wind (kPa)**	**Factored (kPa)**
Roof snow (Ss=1.5 Sr=0.2)		1.40		2.10
Interior suction and positive exterior			0.38	0.15
Roofing / Insulation / Ceiling allowance	0.80			1.00
Steel Deck	0.10			0.13
Total load for Deck Design	**0.90**	**1.40**	**0.38**	**3.38**
Suspended mechanical/electrical	0.20			0.25
Joists at 2000 o/c	0.10			0.13
Total load for Joist Design	**1.20**	**1.40**	**0.38**	**3.75**
W410x60 Girder at 8500 o/c	0.07			0.09
Total load for Girder Design	**1.27**	**1.40**	**0.38**	**3.84**

Factored Joist Area Load = 1.25D + 1.5S + 0.4W
= 1.25 * 1.20kPa + 1.5 * 1.40kPa + 0.4 * 0.38kPa = 3.75kPa

Factored Joist load = Factored area loading * Tributary width
= 3.75kN/m² * 2.0m = 7.50kN/m

Unfactored Snow load on joist = Area live loading * Tributary width
= 1.4kN/m² * 2.0m = 2.8kN/m

Serviceability Snow load for deflection check = (SLS = 0.9) * 2.8kN/m = 2.52kN/m

We will now refer to Table 6.2, which shows the CanAm™ joist catalogue of an 8m span and look for the column with a factored load of 7.50kN/m.

Table 6.2. Factored load chart for 8m span

Span	Joist Depth	Factored load (kN/m) Service load (kN/m)												
		4.5 3.0	6.0 4.0	7.5 5.0	9.0 6.0	10.5 7.0	12.0 8.0	13.5 9.0	15.0 10.0	16.5 11.0	18.0 12.0	19.5 13.0	21.0 14.0	22.5 15.0
(m)	(mm)													
8	400	9.2 87	11.5 85	14.0 85	16.0 85	20.5 85	20.6 83	24.0 85	26.5 88	28.3 84	31.5 84	34.4 84	36.5 88	38.0 88
	450	9.6 86	10.3 70	12.5 70	14.1 68	16.9 65	18.5 58	20.2 85	21.7 85	23.9 85	28.9 84	28.5 85	30.3 85	30.3 82
	500	9.7 107	10.3 84	11.9 78	13.4 73	15.8 70	16.0 68	17.0 74	17.3 69	19.1 69	20.5 69	22.9 85	24.9 87	26.0 89
	550	10.4 131	10.6 98	11.6 85	13.3 84	14.5 80	15.5 80	16.0 76	17.1 81	17.9 77	19.5 77	22.7 77	24.8 76	24.8 73
	600	10.7 158	10.9 117	11.8 98	14.1 101	15.0 91	15.2 89	15.6 92	16.4 91	17.6 89	18.1 84	22.6 81	24.5 83	24.5 82
	650	12.2 200	13.6 200	13.7 176	14.3 151	15.2 128	15.3 110	15.4 98	15.6 96	16.6 104	17.9 98	20.0 92	22.4 89	22.8 84
	700	12.3 200	13.7 200	13.9 200	14.4 176	15.6 156	16.0 128	16.1 114	16.5 106	17.0 106	18.0 112	20.3 109	21.3 102	22.6 107

Lightest joist: See margin for corresponding depth

Source: CanAm™ joist catalogue

The most economical depth is the one with the shading, giving a 550mm deep truss.

Looking closer at that corner of the chart we see values shown in Table 6.3.

Table 6.3. Detail from factored load chart for 8m span

Span	Joist Depth	Factored load (kN/m) Service load (kN/m)			
		4.5 3.0	6.0 4.0	7.5 5.0	9.0 6.0
(m)	(mm)				
8	400	9.2 87	11.5 85	14.0 85	16.0 85
	450	9.6 86	10.3 70	12.5 70	14.1 68
	500	9.7 107	10.3 84	11.9 78	13.4 73
	550	10.4 131	10.6 98	11.6 85	13.3 84
	600	10.7 158	10.9 117	11.8 98	14.1 101
	650	12.2 200	13.6 200	13.7 176	14.3 151
	700	12.3 200	13.7 200	13.9 200	14.4 176

Source: CanAm™ joist catalogue

Two loads are given at the top of the column: (1) the factored load, which we just used, and (2) the "service load," which is the factored load divided by 1.5. The most economical joist is shown in the pink square. The upper number in the pink square is the joist mass. Our most economical joist has a joist mass of 11.6kg/m.

For computing the live load deflection, we use the small number beneath the joist mass; in this case, 86. This represents the percentage of live load to produce a deflection of $L/360$. In this case the live load

listed at the top of column is 5.0kN/m, so the live load that will cause a deflection of L/360 will be 86% * 5kN/m = 4.3kN/m. This is well above our serviceability snow load for deflection check of 2.52kN/m, and we can conclude that serviceability criteria are satisfied.

Result:
Use 550 Deep OWSJ with a mass of 11.6kg/m
Span / Depth = 8m/.550 = 14.5

Compare to rolled section:
Calculating, we would find that the most economical rolled section is

$$\text{Factored Beam Moment} = \frac{WL^2}{8} = \frac{7.5KN/m*(8.0m)^2}{8} = 60KNm$$

Span / Depth of 25 gives: 8,000 / 25 = 320 say W310.
W310x21 has M_r = 89.1kNm. I = 37.0 x 10^6mm^4 mass = 21.1kg/m
Compute deflection under service snow load

$$\text{Service Snow deflection} = \Delta = \frac{5*W_l*L^4}{384*E*I} = \frac{5*(2.52KN/m)*8.0^4*1000}{384*200*37.0} = 18.2mm = L/440$$

Result:
Use W310x21 I = 37.0x106mm4 Mr = 89.1kNm

The OWSJ saves 21.1 – 11.6 = 9.5kg/m, which is about 45% of the weight of the rolled section. However, the depth of the OWSJ is 550mm deep as opposed to the rolled section at 310mm deep.

6.15 COMPUTING THE EFFECTIVE INERTIA OF A JOIST

Some joist catalogues do not list the "effective inertia" of the joist. This information may be required if doing frequency or ponding calculations. The effective inertia can be back-calculated from the service load table. This is the "effective" inertia, as the deflection will also include shear deflection. If using a moment of inertia listed in a joist manufacturer's catalogue for deflection calculations, then the inertia should be decreased by 10% to account for shear deflections. If checking joist deflection based on the equation

$$\Delta = \frac{5wL^4}{384EI},$$

then the value of I based on the area of the chords and their distance from the neutral axis should be divided by 1.1 to account for shear deflections. In some of the earlier S16 editions this was a code clause, but now S16 requires that deflection of joists be "calculated based on all the components of the joists." The S16 procedure of truss analysis might work for the joist designer who has access to all the truss geometry, including the cross-sectional area for chords and diagonals, but it does not work for the building structural engineer at the time of his or her design work as this will be long before the joist supplier has been selected. The building designer will at times require the inertia of the joist for ponding, deflection, vibration and frequency calculations. When doing building design calculations where the inertia of the joist is provided, this inertia should be divided by 1.1 to account for shear deflections.

6.16 JOIST INFORMATION THAT NEEDS TO BE SHOWN ON STRUCTURAL DRAWINGS

The structural designer generally selects and shows the following information on drawings:

- The depth of OWSJ to be used (the weight is NOT supplied by the building designer – this is determined by the joist designer and will depend on the joist system selected).
- The spacing of OWSJ and the layout.
- The supporting structure for the OWSJ, including the girders and the lateral load carrying system.
- The live loading to be used by the OWSJ designer, including any snow build-up requirements.
- Any ponding loading if the ponding loading governs the design.
- The live load deflection requirements (L/300 or L/360).
- The dead load requirements to be used by the OWSJ designer, including superimposed dead loads on roofs.
- Any requirement for extension of bottom chords of trusses required for column stability.
- Any wind uplift and downward wind load requirements (the downward wind load is a required by S16 Clause 16.4.1(a) to be called up on the structural drawings along with other loading that the joist designer will need). The wind uplift is required as Clause 16.5.5.2 requires that, when net uplift due to wind forces is a design requirement, a single line of bottom chord bracing shall be provided at each end of the joist near the first bottom chord panel points. To avoid "extra charges" for placing this bracing, the net wind uplift should be shown on drawings. The downward wind load is a fundamental part of the loading on the joists.
- Any camber requirements to reduce the dead load deflection.
- Any requirements for point loads such as those required to support basketball hoops in gyms.
- Any bottom chord extensions required for stability of columns or beams.

An example of a portion of a structural plan showing the OWSJ layout is shown in Figure 6.13 and the accompanying joist table is shown in Figure 6.14.

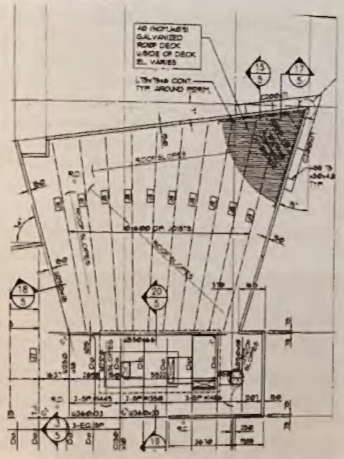


Figure 6.13. Structural plan showing the OWSJ layout

Figure 6.13 shows a sample layout of joists on a structural drawing. The depth of the joist can be determined as can the number and spacing of joists. On structural drawings the joist is shown with a dashed line to distinguish it from rolled beam sections, which are shown as a solid line. The numbers in boxes on the joist refer back to the joist table, a sample of which is shown in Figure 6.14.

OPEN WEB STEEL JOIST LOAD TABLE

OWSJ MARK No.	JOIST DEPTH	SUPERIMPOSED LOADS (UNFACTORED) (kN/m) DEAD	LIVE
J1	400	2.2	3.3
J2	500	1.4	
J3	400	1.9	
J4	600	1.5	4.5
J5	600	1.5	
J6	600	1.5	
J7	600	1.5	
J8	600	1.2	
J9	600	1.2	4.1

JOIST NOTES:

1. THE SELF WEIGHT OF THE OPEN WEB STEEL JOIST SHALL BE ADDED TO THE SUPERIMPOSED LOADS NOTED ON THE SCHEDULE.
2. THE SNOW LOAD PROFILES SHOWN ABOVE ARE APPROXIMATED FOR THE TYPICAL JOIST DESIGN.
3. MAXIMUM DEFLECTION UNDER SPECIFIED LIVE LOADS TO BE LESS THAN SPAN/360 UNLESS OTHERWISE NOTED.
4. OPEN WEB STEEL JOISTS AND BRIDGING TO BE DESIGNED FOR A NET FACTORED UPLIFT OF 1.0 kPa
5. ALLOW FOR 2kN (500 LBS.) POINT LOAD AT ANY LOCATION ALONG THE BOTTOM CHORD FOR GYMNASIUM TRUSSES.

Figure 6.14. Joist table taken from the same structural drawings as partial plan in Figure 6.13

During the site services phase of the project, shop drawings of the OWSJs are submitted for review by the structural engineer of record. These drawings will be sealed by a professional engineer and are often three pages of computer calculations showing input loadings and structural design information on the joist. The designer reviews the shop drawings to determine if the loading used by the OWSJ designer complies with that shown on the structural drawings.

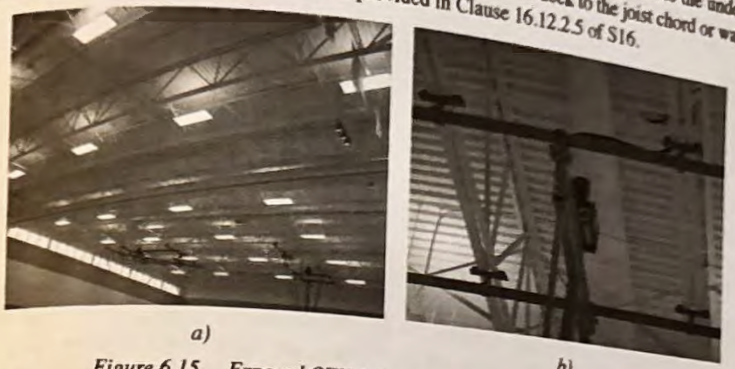
Due to the danger of overstressing chords of OWSJs when attaching heavy pipe racks, and due to the difficulty of attaching effective seismic restraint, OWSJs should not be used for roofs or floors over mechanical rooms. Use rolled sections for framing these areas, and, in addition to the superimposed load from the roof or floor above and any known suspended equipment, add a superimposed dead load of 1kPa to 2kPa to account for the weight of other potential suspended mechanical equipment.

The economy of joists is only appreciated if there is repetition. Avoid using OWSJs and substitute rolled sections in the following cases:

- Where there is only one joist of a given length. (Use rolled section here.)
- Where the joist span is less than 3m.
- Where the total area to be framed is small such that the total number of joists is not significant.

It may be necessary to substitute rolled sections for joists in areas of restricted headroom; for example, to run mechanical ducting down hallway corridors of schools, rolled sections can be substituted for joists with a resulting decrease in structural depth (albeit with a greater weight of structural steel).

Deflection requirements for joists that are next to and parallel to rigid surfaces such as walls will require more stringent deflection limits to prevent excessive slope and movement of the deck at the mid-span of the joist. Under heavy snow loads causing severe joist deflection, in addition to the undesirable steep deck slope between the joist and the wall the connections of the deck to the joist chord or wall trimmer could become compromised. Guidance is provided in Clause 16.12.2.5 of S16.



a) b)

Figure 6.15. Exposed OWSJs forming the roof of high school gym

In Figure 6.15, the trusses have been painted to match the roof deck and the final effect is an attractive structural solution in keeping with the space that it serves. In gymnasiums, OWSJ will often support basketball hoops attached to the chords of the joists. This produces bending in a tension element that must be accounted for by the joist designer (this has been covered in the sample joist notes shown in Figure 6.14). Note that the location of the hoops will often not be known at design time or may move later and all joists for the gym should be designed to accommodate this load.

If it is desired to provide an eyebrow or overhang on the outside of the building, this can be done by specifying joist chord extensions. These cantilevers are designed by the joist supplier provided they are shown and dimensioned on the structural drawings. The joist manufacturer will design the top chord extension to suit the load. In general, this means that the chord will need to be reinforced over the cantilever length and some distance beyond the joist support point, typically beyond the first interior panel point. An interior and exterior view of a joist extension is shown in the two photographs in Figure 6.16.



a) b)

Figure 6.16. a) Interior and b) exterior views of a joist extension

6.17 CAMBER FOR OPEN WEB STEEL JOISTS

Joists are cambered for dead load such that when the dead load is on the joist it does not appear to be sagging, and so that additional ponding of water does not build up on roof joists or wet concrete on floor joists. Unless otherwise specified, the camber requirement is stated in Clause 16.5.13 of S16 as 0.002 of the span length, or L/500; for a 10m joist, this gives a 20mm camber. For floor joists, this is less than the expected dead load deflection (for floors dead loads are often similar to live loads, giving a deflection of L/360). It may be desirable to provide the desired camber on the drawings for longer span floor joists. If camber for joists is too low, then the roof slopes could be compromised and roofs that are sagging under dead load may be more susceptible to ponding loads. Negative cambers are permitted by Clause 16.5.3 for the purpose of controlling roof drainage, but this will require a careful analysis of the roof system.

6.18 BRIDGING FOR OPEN WEB STEEL JOISTS

Bridging is installed between OWSJs to brace the top and bottom chords of the joist for the following reasons:

- To reduce the unsupported length of the bottom chord of the joist. (It would also reduce the unsupported length of the top chord if the joist was supporting a deck or glazing that was not assumed to be a diaphragm.)
- To reduce unsupported compression chord problems caused by wind uplift.
- To assist in erection of the joists by providing top chord stability before the roof diaphragm decking is installed and to assist in plumbing the joists, i.e., holding them in position.

Bridging can be in the form of cross-braces between the joists, or it can be a series of horizontal members tying the bottom chords together with cross-bracing at each end. Clause 16.7.9 requires the spacing of bridging for OWSJs to be dependent on the radius of gyration of the compression flange as follows:

≤ 170*radius of gyration for compression chords

≤ 240*radius of gyration for tension chords

The radius of gyration is taken as that about the plane of the joist web. As the radius of gyration will depend on the type of joist selected by the contractor, structural drawings should state that bridging is required. The spacing of lines of bridging is to be determined by the joist supplier.

Additional bridging will be required in cases of wind uplift, and structural drawings should clearly show the design wind uplift so that this can be taken into account by the joist supplier.

Some clauses that are important and might get overlooked:

> Clause 16.5.7.3 requires that "The factored resistances of the first compression web member subject to transverse shear, and its connections, shall be determined with their respective resistance factors, Ø, multiplied by 0.85." This clause may not be checked in some automated computer steel design programs unless they were custom written to design trusses and joists. When doing design by hand, it would be easier to increase the loads in the first compression diagonal by 1/0.85 and use the normal design charts.
>
> Deflections must now be calculated by truss action, taking account of the additional deflection from the web members. It is no longer suggested that designers take account of shear deflection simply by multiplying the bending deflection by 1.1.

CHAPTER 7
DESIGN OF FLOOR AND ROOF DECKS

An understanding of the design of steel deck for roofs and composite floors is fundamental to the design of structural steel buildings. The majority of the suspended floor and roof surfaces for steel buildings are provided by metal deck, and the spacing of the beams in these suspended levels will depend on the capacity of the deck system. The design of roof and composite decks is frequently omitted from most structural steel courses, and most practising engineers have picked up their knowledge of steel deck design through on-the-job training or by learning from their mistakes. This chapter concentrates on the design of steel floor and roof decks for vertical loading. Chapter 15 deals with the deck as a diaphragm. While selecting a deck and maximum spans from the manufacturer's literature is standard procedure for most designs, there are important issues to review, and the correct interpretation the manufacturer's information will do much to help prevent problems during the construction phase.

7.1 STEEL ROOF DECK

The most common type of steel deck is roof deck, as many buildings will have steel roofs but no suspended floors. Insulation and roofing material are generally added to the top surface of the roof deck to provide a sufficient barrier to energy loss and water ingress.

Steel deck is rolled from flat sheet steel to give a profile that has much more strength and stiffness than was possessed by the flat sheet from which it was rolled. The parts of the deck once it is rolled are shown in Figure 7.1.

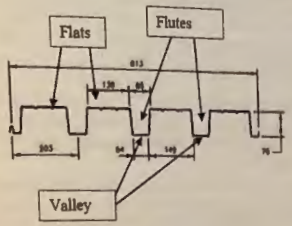
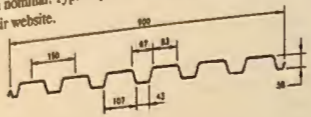


Figure 7.1. Parts of the deck

Although the deck covers a two-dimensional space, the profile of the deck is such that it spans only in the direction of the flutes as a series of "mini-beams." The deck has no ability to transfer gravity loads perpendicular to the flutes.

Standard roof deck is available in 38mm and 76mm, respectively, nominal and sions are soft converted to 38mm and 76mm nominal and 75mm nominal. Typical profiles are as shown in Figure 7.2 for decks produced by VicWest® and shown on their website.



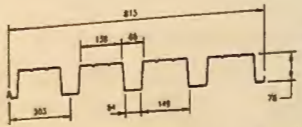


Figure 7.2. Typical roof profiles

The capacity of the deck is computed using the section modulus given in deck tables that are produced by the deck suppliers for the decks that they roll. The deck profiles are usually not symmetric about their neutral axis and due to the calculation methods mandated by Standard S136, the section modulus for mid-span and support conditions will be usually be different. This results in different moment resistance values for positive (tension at the bottom) moments and negative (tension at the top) moments. The moment resistance will be computed as $M_r = Ø*F_y*S$, where the resistance factor is $Ø = 0.9$, the yield stress is F_y = 230MPa, and the section modulus, S, is obtained from the table of properties produced by deck manufacturers, with different values used for support and midspan between supports. A sample section of the table for RD 938 deck is shown in Table 7.1. While the deck panel width is usually less than 1m wide (see the 900mm width of the RD938 in Figure 7.2), the properties given in the table are for a 1m width of deck.

Table 7.1. Moment resistance, yield stress, and section modulus for RD938

Base Steel Nominal Thickness (mm)	Nominal Thickness Z275 Coating (mm)	Mass with Coating (kg/m²)	Section Modulus		Moment of Inertia	Factored Resistance			
						Moment		Reaction	
			Midspan (mm³ x 10³)	Support (mm³ x 10³)	Midspan (mm⁴ x 10⁴)	Midspan (N-m)	Support (N-m)	End (kN)	Interior (kN)
0.76	0.80	8.60	9.59	9.76	213.9	1985.1	2020.3	10.9	15.7
0.91	0.95	10.17	11.70	12.31	266.8	2421.9	2548.2	15.3	22.3
1.22	1.26	13.34	15.87	16.21	364.0	3285.1	3355.5	25.8	38.4
1.52	1.56	16.45	20.00	20.06	454.4	4140.0	4152.4	38.9	58.7
1.91	1.95	—	—	—	—	—	—	—	—

EXAMPLE—MOMENT CAPACITY CHECK ON A DECK SECTION

For 0.91mm thick VicWest® RD938, compute the midspan moment resistance.

$S_{midspan} = 11.70 \times 10^3 \text{ mm}^3$

$M_r = Ø*F_y*S$

$M_r = 0.9 * 230{,}000 \text{kN/m}^2 * 11.70 \times 10^{-6} \text{m}^3$

$M_r = 2.42 \text{kNm}$ (agrees with VicWest® tables: 2421.9Nm = 2.42kNm)

As shown in Table 7.2, there are up to four possible choices for the thickness of standard deck ranging from 0.76mm to 1.52mm. Deck thickness is often referred to by "gauge." The smaller the gauge, the thicker the deck. Gauge is an historic measurement with several standards, so deck thickness should be

Table 7.2. Gauge sizes converted to millimetres and inches

22 Gauge	0.76mm	0.030 inches
20 Gauge	0.91mm	0.036 inches
18 Gauge	1.22mm	0.048 inches
16 Gauge	1.52mm	0.060 inches

Where deck is to be exposed or in institutional projects, the use of 22 Gauge (0.76mm) deck should be avoided as this deck is subject to damage, including possible footprint marks from workers. The use of 16 Gauge (1.52mm) deck is not common as it is more difficult to connect the sheets together at the sides using button punching. As 1.52mm is a less common deck thickness, it is not commonly stocked and delivery times will be longer than the more common thicknesses. It also is not surprising that thicker decks are more expensive as they contain more steel.

The thickness of steel deck designed for vertical loads may have to be increased once the diaphragm design has occurred. The calculation of the forces on diaphragms and the design and detailing of the diaphragm to resist these forces is discussed in Chapter 15.

Deck sections come in strip widths of 900mm or less. In Western Canada, historically the most typical method of connecting the strips of both floor and roof decks was to use button punching, in which the deck sections are crimped together along the side laps. This is still the practice for floor decks where there will be a composite deck-slab that provides much of the diaphragm capacity. For roof decks where there is no topping, the present practice is almost exclusive the use of side-lap screws in high seismic risk areas. The spacing of the side-lap deck connections is a maximum of 600mm, but will usually be more tightly spaced to give the deck sufficient capacity to resist diaphragm loads. It is possible to connect the sides of deck panels by seam welds, but this is not common. Current practice for connection of decks is shown in Table 7.3.

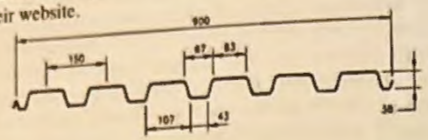
Table 7.3. Typical deck connections

Type of deck	Fastening between sheets	Fastening to structure
Roof deck with no concrete topping	Side-lap screws. Button punching almost never used in seismic risk areas	For 0.76mm and 0.91mm deck usually fastened with pins, self-tapping screws or puddle welding. Puddle welding is becoming less popular in seismic risk areas.
		For 1.22mm and 1.52mm deck sometimes fastened with pins, self-tapping screws or puddle welding. Puddle welding is still popular for 1.22mm and 1.52mm decks even in high seismic risk areas.
Composite floor deck (concrete on composite deck)	Button punching preferred though some designers use side-lap screws	Puddle welding. Occasionally will be fastened with pins if welding is difficult due to weather conditions.

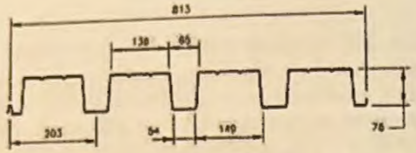
The deck is connected to the supporting structural steel members, usually joists or beams, by a variety of methods including steel pins fired into the deck and the beam using a small explosive charge similar to a bullet being fired from a gun, or through "puddle welding" in which a 19mm diameter hole is burnt in the deck during the welding process and a puddle of weld material holds the deck to the supporting steel.

Dynamic testing of deck sections in 2003 and 2004 has led to concern with the adequacy of button punched side-laps and puddle welded deck connections in roof deck. This has resulted in the formation of a Vancouver committee of concerned engineers from both the deck suppliers and consultants. The concern is in part a result of higher seismic design shears that are required for deck diaphragms designed under

Standard roof deck is available in two depths: 1-1/2 inches and 3 inches. In SI units, these dimensions are soft converted to 38mm and 76mm, respectively, but usually referred to as 40mm nominal and 75mm nominal. Typical profiles are as shown in Figure 7.2 for decks produced by VicWest® and shown on their website.



RD 938



RD 308

Figure 7.2. Typical roof profiles

The capacity of the deck is computed using the section modulus given in deck tables that are produced by the deck suppliers for the decks that they roll. The deck profiles are usually not symmetric about their neutral axis and due to the calculation methods mandated by Standard S136, the section modulus for midspan and support conditions will be usually be different. This results in different moment resistance values for positive (tension at the bottom) moments and negative (tension at the top) moments. The moment resistance will be computed as $M_r = Ø*F_y*S$, where the resistance factor is $Ø = 0.9$, the yield stress is $F_y = 230MPa$, and the section modulus, S, is obtained from the table of properties produced by deck manufacturers, with different values used for support and midspan between supports. A sample section of the table for RD 938 deck is shown in Table 7.1. While the deck panel width is usually less than 1m wide (see the 900mm width of the RD938 in Figure 7.2), the properties given in the table are for a 1m width of deck.

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1.91	1.95	----	----	----	----	----	----	----	----

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$M_r = 2.42kNm$ (agrees with VicWest® tables: 2421.9Nm = 2.42kNm)

As shown in Table 7.2, there are up to four possible choices for the thickness of standard deck ranging from 0.76mm to 1.52mm. Deck thickness is often referred to by "gauge." The smaller the gauge, the thicker the deck. Gauge is an historic measurement with several standards, so deck thickness should be called up in decimal-inch or in millimetre thickness. When deck is called up by gauge, the desired thickness should be included in brackets (e.g., 20Ga (0.036 inches) or 20Ga (0.91mm)).

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The thickness of steel deck designed for vertical loads may have to be increased once the diaphragm design has occurred. The calculation of the forces on diaphragms and the design and detailing of the diaphragm to resist these forces is discussed in Chapter 15.

Deck sections come in strip widths of 900mm or less. In Western Canada, historically the most typical method of connecting the strips of both floor and roof decks was to use button punching, in which the deck sections are crimped together along the side laps. This is still the practice for floor decks where there will be a composite deck-slab that provides much of the diaphragm capacity. For roof decks where there is no topping, the present practice is almost exclusive the use of side-lap screws in high seismic risk areas. The spacing of the side-lap deck connections is a maximum of 600mm, but will usually be more tightly spaced to give the deck sufficient capacity to resist diaphragm loads. It is possible to connect the sides of deck panels by seam welds, but this is not common. Current practice for connection of decks is shown in Table 7.3.

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Composite floor deck (concrete on composite deck)	Button punching preferred though some designers use side-lap screws	Puddle welding. Occasionally will be fastened with pins if welding is difficult due to weather conditions.

The deck is connected to the supporting structural steel members, usually joists or beams, by a variety of methods including steel pins fired into the deck and the beam using a small explosive charge similar to a bullet being fired from a gun, or through "puddle welding" in which a 19mm diameter hole is burnt in the deck during the welding process and a puddle of weld material holds the deck to the supporting steel.

Dynamic testing of deck sections in 2003 and 2004 has led to concern with the adequacy of button punched side-laps and puddle welded deck connections in roof deck. This has resulted in the formation of a Vancouver committee of concerned engineers from both the deck suppliers and consultants. The concern is in part a result of higher seismic design shears that are required for deck diaphragms designed under

current seismic codes. A testing program is underway by Hilti™ and independent testing labs, including academic laboratories in Montreal. While it is early to determine the outcome of this research and committee work, the use of side-lap screws for roof deck has become standard.

Some firms have switched to pin fastening to the structure rather than using puddle welds. Other firms have decided to stay with puddle welding after thermally related issues occurred with roof deck fastened with pins during the summer of 2005. Test results released in November 2009 showed a distinct lack of ductility in mechanical fasteners on heavier (1.22mm and 1.52mm) deck with some of the mechanical fasteners "popping" off the deck as it was cycled through its dynamic loading. The result has been several Vancouver designers electing to use welding between the heavier deck and the supporting beams.

Deck sections come in a maximum length of 12.19m (40 feet). This means that if the designer is relying on having three spans continuous, then the average span length must be less than 4000mm. Note that the product literature of some deck manufacturers will give values for deck capacity with three spans continuous and spans greater than 4000mm.

For ease of handling it is preferred if deck sections are a maximum of 25 to 35 feet long (7.6m to 10.6m). This gives a maximum three-span continuous of 3.5m per span. Deck spans must always end on a supporting member.

7.2 DECK CAPACITY CHARTS

Most designers will use deck capacity charts for determining the suitability and thickness of the deck proposed, as shown in Figure 7.3 taken from VicWest® tables for RD 938 deck.

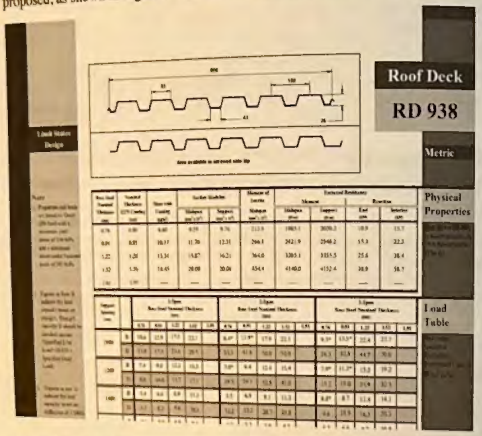


Figure 7.3. Deck capacity chart for RD 938 deck

These charts contain the seemingly conflicting information, as shown in Figure 7.4.

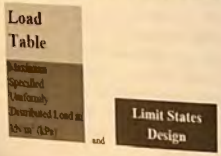


Figure 7.4. Conflicting information in deck capacity chart

The seemingly conflicting terms "maximum specified uniformly distributed load" and "limit states design" on the same chart are not really a contradiction, but are a hangover from the days when all design was done using service loads and allowable stress design. Specified load is an unfactored load: a sum of the dead and live or snow load to be superimposed on the deck. The tables are produced using factored load design methods to determine the capacity of the deck to resist a factored load and then a "specified load" that the deck can resist is determined by dividing the factored load capacity by 1.5. In this way the deck charts are prepared assuming that all load they are carrying is "live load" with a load factor of 1.5. To overcome this restriction it is *theoretically possible* to convert the service load as follows:

$$\text{Service load for tables} = [(\frac{1.25}{1.5} = 0.833) * \text{Service Dead Load}] + \text{Service Live load}$$

The load tables from deck manufacturers are subject to the restriction that the loads should be uniform (deck does not adapt well to point loads) and if values for two-span or three-span conditions are to be used, the spans should be equal and equally loaded. These conditions may be violated in the region of snow drifting.

The Canadian Sheet Steel Building Institute (CSSBI) provides the results shown in Table 7.4 for uniform load w and equal spans.

Table 7.4. CSSBI results for uniform load w and equal spans

	Positive moment	Negative Moment	Deflection	End Support Reaction	Interior Support Reaction
One Span	$\frac{WL^2}{8}$	N/A	$\frac{5WL^4}{384EI}$	0.5 W * L	N/A
Two Span	$\frac{WL^2}{14.3}$	$\frac{WL^2}{8}$	$0.42*\frac{5WL^4}{384EI}$	0.375 W * L	1.25 W * L
Three Span	$\frac{WL^2}{12.5}$	$\frac{WL^2}{10}$	$0.53*\frac{5WL^4}{384EI}$	0.40 W * L	1.10 W * L

This can be expressed graphically as shown in Figure 7.5.

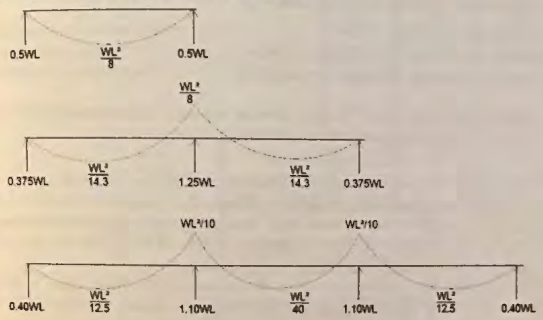


Figure 7.5. Forces resulting from uniform load w and equal spans

This reflects in the charts by the capacity for one, two and three spans, as shown in part in Figure 7.6.

Support Spacing (mm)		1-Span Base Steel Nominal Thickness (mm)					2-Span Base Steel Nominal Thickness (mm)					3-Span Base Steel Nominal Thickness (mm)				
		0.76	0.91	1.22	1.52	1.91	0.76	0.91	1.22	1.52	1.91	0.76	0.91	1.22	1.52	1.91
1000	B	10.6	12.9	17.5	22.1		8.4*	11.9*	17.9	22.1		9.5*	13.5*	22.4	27.7	
	D	13.9	17.3	23.6	29.5		33.3	41.6	50.0	50.0		26.3	32.8	44.7	50.0	
1200	B	7.4	9.0	12.2	15.3		7.0*	9.4	12.4	15.4		7.9*	11.3*	15.5	19.2	
	D	8.0	10.0	13.7	17.1		19.3	24.1	32.8	41.0		15.2	19.0	25.9	32.3	
1400	B	5.4	6.6	8.9	11.3		5.5	6.9	9.1	11.3		6.8*	8.7	11.4	14.1	
	D	5.1	6.3	8.6	10.8		12.2	15.2	20.7	25.8		9.6	11.9	16.3	20.3	

Figure 7.6. Deck capacity chart detail
Source: VicWest®

The definition of span for the deck is the lesser of:

1) The centre to centre spacing on the supports.
2) The clear distance between the supports plus the depth of the deck. This would apply if the deck was sitting on wide-flange beams or a wide wall.

If the spans of the deck are not reasonably equal, or if the loading is not reasonably uniform, it may be necessary to perform an analysis using a plane frame program to get the moments in the deck and to determine the thickness required.

Web crippling occurs when the shear carried by the deck exceeds the capacity of the deck at the support. The web crippling capacity for interior supports is approximately twice the capacity of the web crippling capacity of the deck at exterior supports. This is fortunate as interior supports usually have twice as much load, as they get load from two sides. Other than web crippling, no checks are done on the shear capacity of roof deck or floor deck acting as a form for wet concrete. Extracting the properties from the table for RD 938 we can see the following resistances; the web crippling capacity is given in the "Reaction" columns shown in Figure 7.7.

Base Steel Nominal Thickness (mm)	Factored Resistance			
	Moment		Reaction	
	Midspan (N-m)	Support (N-m)	End (kN)	Interior (kN)
0.76	1985.1	2020.3	10.9	15.7
0.91	2421.9	2548.2	15.3	22.3
1.22	3285.1	3355.5	25.8	38.4
1.52	4140.0	4152.4	38.9	58.7
1.91	-----	-----	-----	-----

Figure 7.7. Web crippling capacity
Source: VicWest®

It is important that a check be made to see which cold-formed steel standard the deck capacity chart was worked out for. The chart should indicate that the latest edition of S136 has been followed, as this is the standard for the design of cold-formed structural steel members and is the standard referenced by S16. The resistance of the decks to web crippling was significantly reduced in S136-01 and this reduction is reflected in more recent editions.

To limit web crippling, the widths of support for the deck should be at least equal to the height of the deck. That is, a 38mm deck should be supported on 38mm wide supports, and 76mm deck should be supported on 76mm wide supports. A slight increase in the web crippling capacity occurs if the supporting element is wider. The requirement of a 76mm bearing length may be a problem when using some lightweight open web steel joists. The CSSBI recommends that to get good puddle welds the thickness of the supporting member should be at least 2.5 times the thickness of the deck. This will be an issue with

thick decks (such as 1.22mm) installed on joists with thin cold-formed chord sections. Minimum base material values are set out in Table 7.5.

Table 7.5. Minimum base material thickness for puddle welding and pins or screws for different deck thickness

Deck Thickness	Minimum base material thickness for puddle welding	Minimum base material thickness for pins or screws
0.76mm	1.9mm	See pin manufacturer's literature
0.91mm	2.7mm	See pin manufacturer's literature
1.22mm	3.1mm	See pin manufacturer's literature
1.56mm	3.8mm	See pin manufacturer's literature

7.3 APPLYING DECK CAPACITY PRINCIPLES TO A ROOF DECK

For a roof located in Vancouver where $C_w = 1.0$, $S_s = 1.8$, and $S_r = 0.2$, examine a region of the roof that has no snow buildup and determine the maximum span for VicWest® 938 x 0.91mm deck. Assume 1kPa roofing weight and a minimum of three spans continuous. Downward wind for this roof is computed to be 0.5kPa.

1) Using the data in Table 7.6, compute the load sheet for the deck. Note that downward wind is included and 0.4 times the downward wind is included under "Service Load."

Table 7.6. Uniform roof snow loading – Vancouver snow load

Load Source	Dead (kPa)	Snow (kPa)	Downward Wind (kPa)	Service Load (kPa)	Factored Load (kPa)
Snow Load (Ss=1.8 Sr=0.2)		1.64		1.64	2.46
Downward Wind			0.50	0.20	0.20
Roofing / Suspended Mech / Ceiling	1			1.00	1.25
Deck	0.1			0.10	0.13
Total load for Deck Design	**1.1**	**1.6**	**0.5**	**2.94**	**4.04**

2) The simple and most common approach is to use VicWest® tables for 938 deck and find the point where a three-span, 0.91mm deck can no longer support an allowable load of 2.94kPa: Table 7.7 has been reduced to show only three span continuous and only in the range of span of interest.

Table. 7.7. VicWest® tables showing service load for three span continuous

Support Spacing (mm)		3-Span Base Steel Nominal Thickness (mm)				
		0.76	0.91	1.22	1.52	1.91
2200	B	2.8	3.5	4.6	5.7	
	D	2.5	3.1	4.2	5.2	
2400	B	2.3	2.9	3.9	4.8	
	D	1.9	2.4	3.2	4.0	
2600	B	2.0	2.5	3.3	4.1	
	D	1.5	1.9	2.5	3.2	

What we see is that for "B" (Bending), a service load of 2.9kPa can be carried at a span of 2400mm. The "D" (Deflection) can withstand a load of 2.4kPa for a deflection of 1/240 of span. If we wish L/360 the allowable load for deflection must be divided by 1.5 giving a maximum snow load of 1.6kPa for L/360. This is greater than our snow load for deflection calculations of 0.9*1.64kPa = 1.48kPa and the maximum span is governed by bending and is 2400mm.

3) As an alternate procedure, use the section property values in the tables

a) Negative (support) moment capacity, continuous over three spans

$M_{neg} = \frac{WL^2}{10} = M_r = 2.548$kNm/m. With w = 4.04kPa*1m = 4.04kN/m, this produces

$$\frac{4.04 kN/m * L^2}{10} = 2.548 kNm/m \rightarrow \mathbf{L = 2.51m}$$

b) Positive (midspan) moment capacity, continuous over three spans

$M_{pos} = \frac{WL^2}{12.5} = M_r = 2.432$kNm/m. With w = 4.04kN/m, this produces

$$\frac{4.04 kN/m * L^2}{12.5} = 2.421 kNm/m \rightarrow \mathbf{L = 2.74m.}$$

c) Web crippling, maximum factored resistance to reaction at interior support = 22.3kN.

Maximum factored reaction = 22.3kN = 1.10(w*L). With w = 4.04kN/m, this produces

$$L = \frac{22.3 kN}{1.10 * 4.0 kN/m} = 5.06m \rightarrow \mathbf{L = 5.06m}$$

Result:
Providing the deck is three spans continuous, the governing result is that L_{max} = 2.51m controlled by bending at the support

Check Deflection under SLS*Snow load:

$$\Delta = (0.53) * \frac{5 * (1.64 * 0.9) * (2.51m)^4}{384 * 200 * 0.267} * 10^3 = 7.57mm$$

7.57mm = L/330

Solve for L to get L/360 and find L = $\sqrt[3]{\frac{384 * 200 * 0.267}{360 * 0.53 * 5 * (1.64 * 0.9)}} = 2.44m$

Governing span = 2.44m

Note that downward wind is ignored in the calculation of deflection – this is a reasonable assumption for Vancouver, but if your project is in an area where drifting and wind can be expected due to the length of the "real" winter you might want to reconsider this simplifying assumption.

7.4 DECK CAPACITY CHARTS USING FACTORED LOADS

One of the questions that comes up when designing steel decks for projects in Canada is, "why don't the charts have factored loads?" Almost all of our structural design is based on factored loads and deflections are usually based on a limit of L/360. When we look at deck charts, the "service" loads given need to be multiplied by 1.5 to get factored load resistance and the load to produce a deflection of L/240 needs to be divided by 1.5 to produce a deflection of L/360. The reason that the deck manufacturer's state that allowable loads are used is that the decks are sold into both the U.S. and Canadian markets and allowable stress design is more common in the U.S. The manufacturer's also state that designers are more familiar with the use of allowable loads for design of decks – this may have been true a couple of decades ago but most engineers graduating now in Canada are much more familiar with limit states design and regard "allowable load" design as an anachronism. Going to factored load deck design tables to bring them into line with current limit states design practice might be one solution, but it might also be worthwhile considering design charts that present the information graphically, as shown in Figure 7.8.

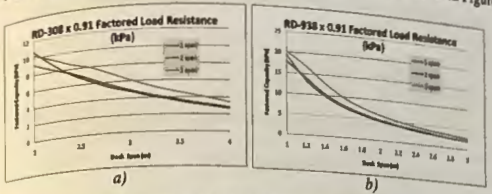


Figure 7.8. Factored load capacity for: a) 38 x 0.91 and b) 76 x 0.91 deck showing the effect of using one, two, or three spans continuous

EXAMPLE—ROOF DECK IN HIGH SNOW AREA WHERE THE LOADING EXCEEDS TABLE CAPACITY

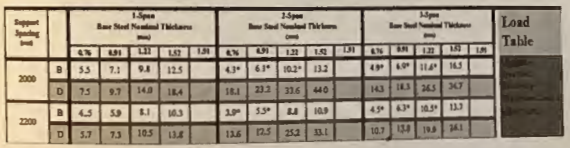
Using the data shown in Table 7.8, design a roof deck for a mid-station of a ski resort in a high snow area with S_s = 8.6, S_r = 0.5, and downward wind = 1kPa. Using the RD308 profile, select the maximum span for a 76 x 0.91 deck for a flat roof. The design dead load of roofing and deck is 1.1kPa. No snow build-up due to drifting. Roof size is 30m x 50m.

Table 7.8. Uniform roof snow loading – resort area

Load Source	Dead (kPa)	Snow (kPa)	Downward Wind (kPa)	Service Load (kPa)	Factored Load (kPa)
Snow Load (Ss=8.6 Sr=0.5) (I_s=1.0)		7.38		7.38	11.07
Downward Wind			1.00	0.40	0.40
Roofing / Suspended Mech / Ceiling	1			1.00	1.25
Deck	0.1			0.10	0.13
Total load for Deck Design	**1.1**	**7.4**	**1.0**	**8.88**	**12.85**

Looking at the RD308 chart with a service load of 8.88kPa, we quickly see that the charts shown in Table 7.9 do not go to this high a load unless we go to 1.22 deck. The highest bending capacity at the lowest span given of 2000mm is 7.1kPa and that is only for one span continuous.

Table 7.9. VicWest® tables showing support spacing



Stay with our problem definition of 0.91mm and go to first principles.

Treat as three spans continuous, factored load = 12.9kPa

For web crippling in three spans condition, $P_f = 1.10\ wL$, Resistance = 22.7kN

- 1.10* 12.9kN/m * L = 22.7kN
- Solve for L get L = 1.59m

For negative (support) moment with three spans continuous and a factored load of 12.9kPa

- For interior support moment = $wL^2/10$
- Resistance = 5.572kNm
- Solve for L get
- L = 2.08m (negative bending)

For positive (midspan) moment with three spans continuous with a factored load of 12.9kPa:

- Continuous over three spans $M_{pos} = \frac{WL^2}{12.5} = M_r = 5.322\text{kNm/m}$.
- With w = 12.9kN/m, this produces $\frac{12.9kN/m*L^2}{12.5} = 5.322kNm/m$ → L = 2.27m.

Governing L = 1.59m (web crippling) ← Governs for three spans continuous

Repeat for two spans continuous with a factored load of 12.9kPa

- Check Web Crippling:
- For web crippling $P_f = 1.25\ wL$
- Resistance = 22.7kN
- 1.25* 12.9kN/m * L = 22.7kN
- Solve for L and get L = 1.40m

For negative moment in two spans continuous condition with a factored load of 12.9kPa

- For interior support moment = $wL^2/8$
- Moment resistance support = 5.572kNm
- Solve for L get: $12.9\text{kPa}*(L^2)/8 = 5.572\text{kNm}$ → **L = 2.85m.**

For positive moment with two spans continuous with a factored load of 12.9kPa:

- Continuous over two spans $M_{pos} = \frac{WL^2}{14.3} = M_r = 5.322\text{kNm/m}$.
- With w = 12.9kN/m, this produces $\frac{12.9kN/m*L^2}{14.3} = 5.322kNm/m$ → **L = 2.51m.**

Result: L = 1.40m (Web crippling governs in this case for two spans continuous).

Repeat for single span condition with factored load of 12.9kPa:

For web crippling in 1 span condition $P_f = 0.5wL$ Resistance at end = 14.3kN

- 0.5* 12.9kN/m * L = 14.3kN
- Solve for L get L = 2.22m

$$\frac{wL^2}{8} = \frac{12.9*L^2}{8} = 5.322KNm$$

Positive Moment: Solve for L get 1.81m

Deflection checks were done by looking at the chart and noting that the capacity for deflection at 2000mm was at least 9.7kPa, which is greater than our SLS*Snow load of 0.9*7.38kPa = 6.64kPa so deflection does not govern in this case. In the deck tables the deck deflection capacity "D" is the load that when applied to the deck gives a deflection of L/240. Some structural designers feel that a deflection criterion of L/240 is too bouncy and would prefer L/300 or L/360. As the deflection is linear in terms of load, the load to get L/360 is D*(240/360) and the load to get L/300 is D*(240/300). Note that L/360 is less deflection than L/240, so for a given span and deck stiffness it will take less load to get to that deflection.

Results with the governing condition highlighted are shown in Table 7.10.

Table 7.10. Governing conditions

	Web Crippling	Moment Capacity (negative moment)	Moment Capacity (positive moment)	Govern
3 spans continuous	**1.59m**	2.08m	2.27m	1.59m
2 spans continuous	**1.40m**	1.85m	2.51m	1.40m
Single span	2.22m	N/A	**1.81m**	1.81m

Result:
If you are certain that a three-spans-continuous layout can be achieved, use 1.59m spans (round to 1.5m) but otherwise use 1.4m as the maximum span.

7.5 CALLING UP ROOF DECK ON STRUCTURAL DRAWINGS

Note that the following items are shown on the drawings:

- The direction of the deck flutes.
- The depth of the deck (40mm Nominal or 75mm Nominal).
- The thickness or gauge of the deck (if gauge is used, the desired thickness should be called up after).
- That the deck is galvanized.
- Where possible deck elevations or the fact that the deck is sloped should also be shown on plan.

Details such as the requirement of having three spans continuous, spacing of button punching and puddle welding requirements will generally be given in general notes. A sample detail is shown in Figure 7.9.

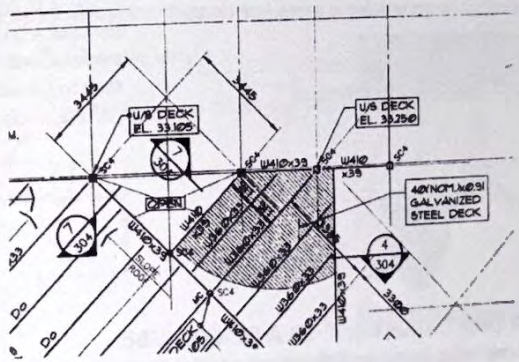


Figure 7.9. When calling up roof deck on drawings, the height of the deck (40mm or 75mm), the thickness of the deck (in this case 0.91mm), and the direction of the deck flutes must all clearly be shown. Note that there is sufficient information to determine the elevation of the top of the steel members

When designing steel roof deck from first principles, the deck has traditionally been designed using allowable stress and service loads and comparing to a maximum stress of $0.6F_y$ (with 230MPa steel this works out to 132MPa). The preferred method now is to use factored design and factored loads and compare to $0.9F_y$. The moment resistance of the deck is therefore $Ø*F_y*S$. (Note that for deck, Z, the plastic section modulus, is never given and never used.)

As each manufacturer will have slightly different rolling profiles, steel decks with similar depths may have quite different properties. The capacity of the deck produced by one manufacturer may vary by up to 15% when compared to the capacity of the deck produced by another manufacturer, even though the two decks are essentially similar in thickness and overall depth. The resistance for negative bending moments is often quite different from the resistance for positive moments. Care should therefore be taken when specifying a deck when the loads are close to the maximum capacity of that deck, as the contractor may choose deck from a different manufacturer. If necessary, minimum values of M_r at the support and at midspan should be specified.

7.6 DESIGN OF ROOF DECK FOR SNOW DRIFTING

In Chapter 2, the method of computing snow drifting loads was studied with the following results for the snow loads around a mechanical penthouse or other roof step. The loads at the step are highest near the wall and taper off to the standard roof snow load as we move away from the step. Two conditions exist for decking adjacent to the wall.

1) The deck flutes run parallel to the roof step. (The support beams are perpendicular to the roof step.) In this case, the deck flutes closest to the wall are subjected to the maximum snow loading.
2) The deck flutes run perpendicular to the roof step. (The support beams are parallel to the roof step.) In this case, the deck flute will experience a decreasing load as we move away from the wall as shown in Figure 7.10.

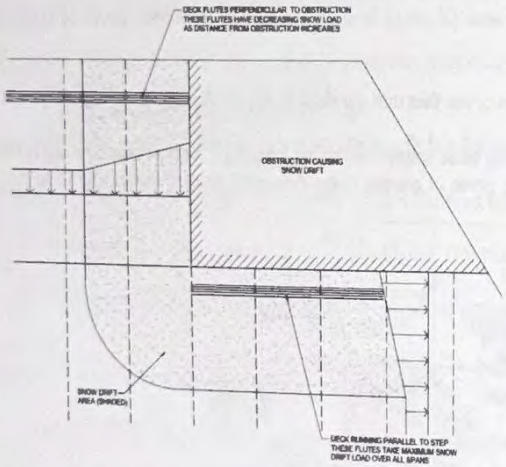


Figure 7.10. Two possible conditions for deck direction in a snow drift area. It may be possible to get higher spans in the case where the deck flutes are perpendicular to the obstruction than in the case where they are parallel to the obstruction.

EXAMPLE—SNOW DRIFT ADJACENT TO A PENTHOUSE

A 4m high mechanical penthouse sits on the 40m x 40m roof of a normal importance building to be located in an area with $S_s = 1.8$ and $S_r = 0.3$. The mechanical penthouse is 7m x 5m. The snow load adjacent to the penthouse is shown in the diagram in Figure 7.11 and drift area snow loading is shown in Table 7.11. Determine the maximum spacing of supports for RD938x0.91 deck. The dead load on the deck is 1kPa.

Result:

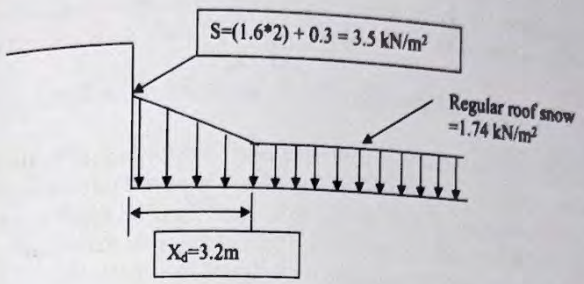


Figure 7.11. Snow load adjacent to the penthouse

Table 7.11. Drift area snow loading

DRIFT AREA SNOW LOADING			
Load Source	**Dead (kPa)**	**Live (kPa)**	**Factored (kPa)**
Drift Snow Load adjacent to 7m x 5m penthouse (S_s=1.6 S_r=0.3)		3.5	5.3
Roofing / Suspended Mechanical / Ceiling	1.0		1.3
Deck	0.1		0.1
Total load for Deck Design	**1.1**	**3.5**	**6.6**

Note that under local Vancouver practice, downward wind is ignored in the drift areas.

The worst case is where the deck is spanning parallel to the building step and examining the tables for three spans continuous, this loading is satisfied with a maximum span of 1.8m and 2.0m, while deck segments acting as a simple span would have a maximum span of about 1.7m. (It would also be possible to use $(1.25/1.50)*1 + 3.5\text{kN/m}^2 = 4.33\text{kN/m}^2$, giving a maximum span of slightly below 2.0m if three spans continuous, and about 1.7m if using the deck in a location where it is in a single span condition. Note that these changes are not sufficient to warrant the additional calculation of reducing the dead load by 0.833).

The other situation is where the deck is running perpendicular to the building step as shown in Figure 7.12. Examining the factored loading diagram and doing a quick analysis we find the following results:

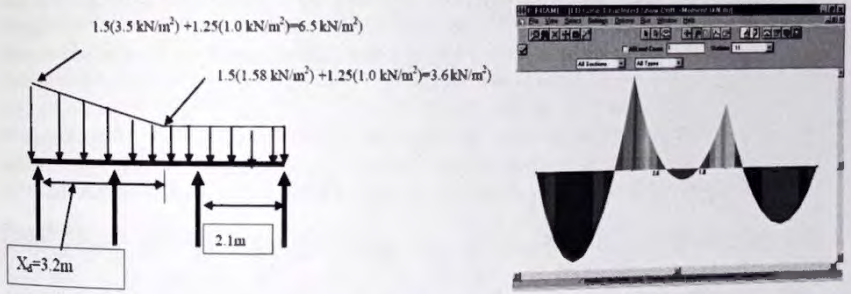


Figure 7.12. Effects of snow drifting when deck flutes are perpendicular to the obstruction

Comparing the results for a 2.1m deck span we find that factored negative moment and factored positive moment are all within the capacity of the deck. The foregoing procedure would be used to evaluate a situation where it is difficult to locally change the deck gauge or support spacing. The typical situation would be to design the deck using the method shown for deck flutes parallel to the wall.

Deck is protected by a zinc coating (galvanized). Z275 finish is a heavier finish used on a small number of jobs where the underside of the deck is exposed to the elements, high humidity or in parking lot applications. Wipe-coat finishes (Z075) are used on most jobs where excessive corrosion conditions are not present.

The deck is puddle welded to supporting members and members parallel to the deck span for the following reasons:

1) So that the supporting member has a positive connection to the deck, as the deck is the diaphragm stabilizing the supporting member. Unless specifically requested on drawings the deck installers will not fasten to members that are parallel to the deck span. This results in these members not having their top flanges laterally supported. To avoid this issue diaphragm notes should clearly state that the deck is to be fastened to the top flange of all members that are in contact with the deck including those members that are parallel to the deck span.
2) So that uplift forces from the wind can be transferred to the supporting member.
3) So that the deck diaphragm capacity is improved; deck diaphragm capacity is highest when all flutes of the deck are puddle welded to supporting members.
4) In the case of floor deck, to help provide some composite action to reduce vibration. Puddle welding of the deck does not provide sufficient shear transfer to consider the slab as composite for strength calculations but it is sufficient to help reduce vibrations.

VicWest® catalogues sometimes list two types of 38mm deck: RD938 and RD38. RD38 is no longer available and properties for RD938 should be used. (A similar comment can be made about floor deck with HB938 and HB38 – use HB938).

Acoustic deck is sometimes specified for use where it is desired to soften some surfaces to reduce noise levels. Acoustic deck works by having many holes punched in it after the zinc coating process has completed. After the deck is installed, an insulating material is placed within the deck ribs as a sound absorber. Because the deck holes must work with insulation behind to absorb tbe sound, acoustic deck is never used for floor deck. The holes and backing insulation absorb sound waves instead of reflecting them, and the result is a lowering of the sound level. There are, however, a few drawbacks with the use of acoustic deck from the designer's standpoint.

- The first of these is the holes reduce the capacity of the deck by about 10% and this should be included in the calculations when choosing the deck gauge and support spacing.
- The second structural drawback is that acoustic deck is hard to get in thicknesses greater than 0.91mm and the availability should be checked prior to specifying acoustic deck with greater thickness.
- The third drawback is that as the holes are added after the zinc coating is applied, the edges of the metal at the holes are exposed. (The holes are small enough that they would be plugged if galvanizing was applied after the holes were added). This lowers the corrosion potential of the deck. For this reason, acoustic deck should never be specified for outside or for corrosive environments such as swimming pools.
- Acoustic deck connections to supporting members will be weakened if the bottom flange of the deck is also perforated by the noise-reducing holes. If the bottom of the flute is perforated, screwing down of the deck is not recommended and puddle weld capacity will be reduced.

A general rule of thumb for most buildings is that areas of the roof not subject to snow build-up should have a maximum span of 2000mm for 38mm deck and 2500mm to 3000mm for 76mm deck.

One way around a local deck overstress is to use a double layer of roof deck. The capacity will then be the sum of the two individual sheets acting independently (there is no composite action considered between the two sheets). Not all deck sections have the ability to nest together. It may be necessary to remove the nesting lips of the upper deck so that it will fit over the lower deck. If possible, the bottom layer should be installed in an inverted manner to allow for some air space between the two sections so that any moisture that gets in during the installation can dry. Only the top sheet is button punched so the addition of a second sheet does not assist with diaphragm capacity. While this procedure is an option it is seldom done except in a retrofit situation. Usually deck overstresses are handled by adding extra structural members to decrease the span of the deck or locally increasing the gauge of the deck.

7.7 REINFORCING ROOF DECK AT SMALL OPENINGS

CSSBI (CSSBI 10M-06) recommendations for roof deck reinforcement for holes are as shown in Table 7.12 and as illustrated in Figure 7.13.

Table 7.12. CSSBI recommendations for roof deck reinforcement for holes

Hole size: Up to 150mm	No reinforcement necessary provided that not more than 2 webs cut.
Hole size: 150 to 300mm	Use minimum 50x50x6 angle perpendicular to flutes; weld to minimum of 2 flutes each side of opening. (Angle is just supported by adjacent flutes and does not go to supporting structural member).
Hole size: 300 to 450mm	"Provide suitable reinforcement based on a structural analysis of the loads provided". As the deck supplier is to provide the analysis this may be open to interpretation, provide a detail on structural drawings where necessary.
Hole size: Over 450mm	Structural steel solution by the structural engineer. Holes of this size should be reinforced on drawings even if this reinforcement is in the form of a note



Figure 7.13. Local reinforcement of roof deck has been installed at a roof drain. The reinforcement of such holes is covered by the CSSBI rules from Table 7.12.

7.8 DESIGN OF COMPOSITE FLOOR DECKS

Steel floor deck is used with concrete topping. The concrete topping may be the wearing surface of the deck or a wearing surface of tile or carpet may be added afterward.

Similar to roof deck, floor deck comes in two depths: 1-1/2 inches or 3 inches. The profile of steel floor deck is often similar to that of the manufacturer's roof deck, but floor deck has added indentations ("embossments") to make the floor composite. The steel deck acts as formwork for the wet concrete and as positive reinforcement for the composite slab. A composite floor deck ready for concrete placement is shown in Figure 7.14.

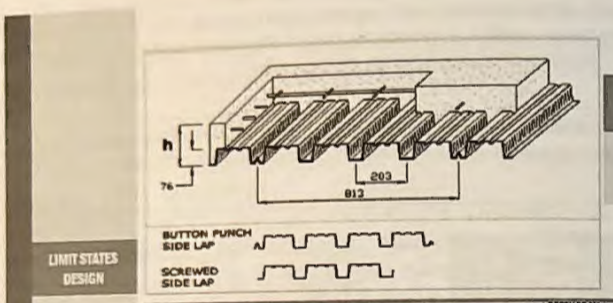


Figure 7.14. Composite floor deck ready for concrete placement. The rebar is in place and chaired. The deck is clean and free of standing water and any dirt, sawdust or construction debris. The deck fastening and installation is preferably reviewed prior to the reinforcing steel being placed. Note the indentations in the deck to make it act in a composite manner

Design and Construction of Composite Floor Systems by Chien and Ritchie outlines the following procedure for designing composite floor deck:

1) Check deflection under fresh concrete including ponding (accumulation of concrete due to deck deflection). A deflection of L/180 or 20mm is a normal limitation.
2) Check effects due to construction load during slab pouring.
3) Check effects due to concentrated construction load.
4) Check shear–bond capacity of composite section.
5) Check maximum concrete compressive stress in composite section.
6) Check maximum steel deck tensile stress in composite section.
7) Check live load deflection of composite section.

Fortunately, these steps have been included in the manufacturer's literature as shown in the Figure 7.15 from VicWest® for the HB308 deck.



Note

1. Load Tables are based on the design of ONE-WAY composite slabs carrying uniformly distributed loads on a simple span basis. For complete design criteria see the VICWEST Hi-Bond Composite Floor Designer's Manual.
2. A uniform loading in excess of 10 kPa (200 psf) is often an indication of concentrated or moving loads. Such conditions may require additional reinforcing steel. Contact VICWEST for additional design information
3. Slab Thickness h, is from underside of steel deck to top of concrete. Maximum span is not to exceed 32h.
4. Properties and loads are based on Grade 230 steel (Grade 33 steel) with a minimum yield stress of 230 MPa (33,000 psi) and a maximum stress under Factored loads of 207 MPa (29,700 psi).

NOMINAL CORE THICKNESS (mm)	AREA OF STEEL (mm²)	MASS WITH ZF75 GALVANNEAL (kg/m²)	SECTION MODULUS MIDSPAN S_m (mm³ x 10³)	SECTION MODULUS SUPPORT S_s (mm³ x 10³)	Moment of Inertia MIDSPAN I_m (mm⁴ x 10⁶)	Moment of Inertia FULL I_f (mm⁴ x 10⁶)	DEPTH FROM NEUTRAL AXIS TO BOTTOM OF DECK Y_b (mm)	MAXIMUM FACTORED REACTIONS EXTERIOR (kN)	MAXIMUM FACTORED REACTIONS INTERIOR (kN)
0.76	1262.7	10.22	19.87	21.24	928.6	1167.1	47.90	10.2	16.2
0.91	1516.8	12.24	25.71	26.92	1192.8	1400.7	48.01	14.3	22.7
1.22	2017.8	16.18	35.34	38.57	1724.2	1860.2	48.23	24.1	38.2
1.52	2523.2	20.23	45.23	47.93	2260.1	2321.9	48.45	36.2	57.5

PHYSICAL PROPERTIES STEEL PROFILE (PER METRE WIDTH) In accordance with CSA Specification S136-07

SLAB THICKNESS h(mm)		141		151		166		176	
Slab Weight, W_c(kN/m²)		2.14		2.36		2.70		2.93	
Concrete Volume, (m³/m²)		0.087		0.097		0.112		0.122	
Base Steel Nominal Thickness (mm)		I_c	d	I_c	d	I_c	d	I_c	d
0.76		11543	93.1	14155	103.1	18825	118.1	22482	128.1
0.91		12333	93.0	15114	103.0	20080	118.0	23964	128.0
1.22		13764	92.8	16849	102.8	22353	117.8	26652	127.8
1.52		15069	92.6	18429	102.6	24426	117.6	29106	127.8

PHYSICAL PROPERTIES COMPOSITE SLAB (PER METRE WIDTH) Composite Moment of Inertia, I_c (mm⁴ x 10⁶) Effective Depth, d (mm)

Base Steel Nominal Thickness (mm)	Span (mm)	1 Span	2 Span	3 Span	1 Span	2 Span	3 Span	1 Span	2 Span	3 Span	1 Span	2 Span	3 Span
0.76	2200	14.6	14.6	14.6	16.2	16.2	16.2	18.5	18.5	18.5	20.0	20.0	20.0
	2400	12.5	12.5	12.5	13.9	13.9	13.9		15.9	15.9		17.2	17.2
	2600		10.9	10.9		12.0	12.0		13.8	13.8			15.0
	2800		9.6	9.6		10.6	10.6						
0.91	2400	14.6	14.6	14.6	16.2	16.2	16.2	18.5	18.5	18.5	20.0	20.0	20.0
	2600	12.7	12.7	12.7	14.0	14.0	14.0	16.1	16.1	16.1	17.5	17.5	17.5
	2800	11.1	11.1	11.1	12.3	12.3	12.3		14.1	14.1		15.3	15.3
	3000	9.9	9.9	9.9		10.9	10.9		12.5	12.5			13.6
	3200		8.8	8.8			9.8						
	3400												
	2600	16.4	16.4	16.4	18.2	18.2	18.2	20.0	20.0	20.0	20.0	20.0	20.0
	2800	14.4	14.4	14.4	15.9	15.9	15.9	18.3	18.3	18.3	19.8	19.8	19.8
	3000	12.7	12.7	12.7	14.1	14.1	14.1	16.2	16.2	16.2	17.5	17.5	17.5

LOAD TABLE (PER METRE WIDTH) Maximum Specified Uniformly Distributed Load in kN/m² (kPa)

Shear Bond Coefficients k_1 = 203.1197 k_2 = 2.3277 k_3 = 0.0510 k_4 = 0.0486

Figure 7.15. Composite floor deck design chart

Lots of useful information is provided on this sheet, including the weight of the slab, the ability of the deck section to act as a form, the capacity of the composite deck system once in service, and several other properties that are required in design of both the deck and the main structural system. As with roof deck, the capacity of the deck both as a form and in its final condition are computed using limit states methods but the loads are provided as an allowable load. A closer look at the capacity section of the HB 308 chart is shown in Figure 7.16.

SLAB THICKNESS h(mm)		141		151		166		176	
Slab Weight, W_c(kN/m²)		2.14		2.36		2.70		2.93	
Concrete Volume, (m³/m²)		0.087		0.097		0.112		0.122	
Base Steel Nominal Thickness (mm)		I_c	d	I_c	d	I_c	d	I_c	d
0.76		11543	93.1	14155	103.1	18825	118.1	22482	128.1
0.91		12333	93.0	15114	103.0	20080	118.0	23964	128.0
1.22		13764	92.8	16849	102.8	22353	117.8	26652	127.8
1.52		15069	92.6	18429	102.6	24426	117.6	29106	127.8

PHYSICAL PROPERTIES COMPOSITE SLAB (PER METRE WIDTH) Composite Moment of Inertia, I_c (mm⁴ x 10⁶) Effective Depth, d (mm)

Base Steel Nominal Thickness (mm)	Span (mm)	1 Span	2 Span	3 Span	1 Span	2 Span	3 Span	1 Span	2 Span	3 Span	1 Span	2 Span	3 Span
0.76	2200	14.6	14.6	14.6	16.2	16.2	16.2	18.5	18.5	18.5	20.0	20.0	20.0
	2400	12.5	12.5	12.5	13.9	13.9	13.9		15.9	15.9		17.2	17.2
	2600		10.9	10.9		12.0	12.0		13.8	13.8			15.0
	2800		9.6	9.6		10.6	10.6						
0.91	2400	14.6	14.6	14.6	16.2	16.2	16.2	18.5	18.5	18.5	20.0	20.0	20.0
	2600	12.7	12.7	12.7	14.0	14.0	14.0	16.1	16.1	16.1	17.5	17.5	17.5
	2800	11.1	11.1	11.1	12.3	12.3	12.3		14.1	14.1		15.3	15.3
	3000	9.9	9.9	9.9		10.9	10.9		12.5	12.5			13.6
	3200		8.8	8.8			9.8						
	3400												

LOAD TABLE (PER METRE WIDTH) Maximum Specified Uniformly Distributed Load in kN/m² (kPa)

Shear Bond Coefficients k_1 = 203.1197

Figure 7.16. Detail of floor deck design chart

The determination of the deck capacity from these charts requires the following steps:

1) Determine the thickness of the slab, which includes the topping and the deck itself. For a 76mm deck with a minimum 65mm topping (63mm or 2.5 inches in imperial units), the minimum slab thickness will be the sum of these or 141mm. The capacity for a composite slab of 141mm thickness is given in the column near the left of the chart. The most common thicknesses of slab to use are the minimum concrete topping of 65mm, giving 141mm overall slab thickness, and 115mm of topping, giving 191mm overall slab thickness. The 115mm topping is often chosen as a two-hour fire rating can be achieved without having to spray the underside of the deck with fireproofing. Unfortunately, the deck charts usually do not provide results for this 191mm slab thickness and results must be inferred from the 176mm slab thickness.
2) The deck gauge is given at the extreme left – the usual choice being 0.91mm deck.
3) The span of the deck is determined from the layout of beams in the conceptual design. Let's say 3000mm was selected for the deck span. The permitted superimposed load for the composite deck then would be 9.9kPa. This is for the allowable superimposed load – the charts have allowed for the weight of the slab (as shown in the note at right) so the designer has to compare the specified load (usually Live Load + Partition Loading) to the capacity given.
4) The chart shows several spans where no capacity value has been provided; for example, no capacity is provided for a 0.91mm deck with a 176mm slab thickness in a 3200mm span condition. This is because the deck needs to be shored for concrete placement where the values are not given and it is usually not economical to shore metal deck for concrete placement (extra labour and obstruction to construction activities below). Contractor expectation is that you have designed your deck to have the concrete placed without shoring, and if shoring is required it should be

clearly stated on the drawings to avoid extra charges. If using 115mm of topping you should check that the deck works as a form by hand calculations.

5) The 1-Span, 2-Span and 3-Span columns on the composite deck charts refer to the system when the deck is acting as a form for wet concrete and construction live load. The deck supporting wet concrete and acting as a form is better able to carry loads if it is in a three-span condition in the same way that roof deck is able to carry loads better in a three-span configuration.
6) The load capacity of the composite deck section is not dependent on the number of spans – the composite deck is assumed to be a series of simple span concrete beams with the deck acting as the reinforcing steel.
7) The weight of the slab is already included in the deck charts so the dead load of concrete does not need to be included in the load calculation when looking at the deck charts.

A comparison of the governing criteria for roof and floor deck in its complete state is given in Table 7.13.

Table 7.13. Comparison of criteria for rood and floor deck

	ROOF DECK	FLOOR DECK
Moment Capacity	Differential stress between top and bottom of deck section giving positive and negative moment capacity.	Compressive stress in concrete, tension in deck. Gives positive moment capacity only. **Treat as simple span.**
Shear Capacity	Carried by webs of flute, governed by web crippling.	Carried by shear capacity of deck reinforced concrete. Governed by shear bond.
Deflection	Deflection calculated using the moment of inertia of deck section, can take advantage of continuity.	Deflection is based on a composite moment of inertia and simple span deflection but in practice is seldom checked. Keep the span / depth of the slab less than 32.

Graphically, the representation of the different stress regimes in the deck is shown Figure 7.17.

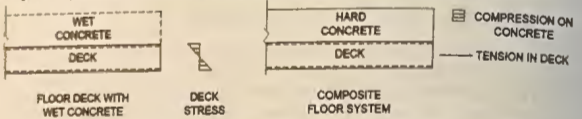


Figure 7.17. Change in the stress regime of the deck when it goes from acting as a form for the wet concrete to a composite section with the hardened concrete. When acting as a form for the wet concrete the deck is a bending element, but when in the composite condition the deck is acting as a tension element and is equivalent to reinforcing bars in a concrete slab

The composite deck system can carry much more load than the deck acting alone, as the full deck section is in tension and the lever arm between the centroid of compression and tension forces is much larger in the composite system.

7.9 COMPOSITE STEEL FLOOR DECK DESIGN

Verify that VicWest® HB308 (75mm deck) with 0.91mm thickness and ZF75 Galvanizing and 65mm cover slab can be used for 3.2m spans. The deck is designed for a live load of 4.8kPa and a partition / mechanical load of 1.0kPa. The deck will be placed in a three spans continuous configuration.

Solution:
The first and easiest solution is to examine the VicWest® tables and examine the load capacity listed.

75mm Deck + 65mm Topping = Overall thickness of 140mm
Span / Depth = 3200 / 140mm = 22.9 (OK <32)
Determine the specified load:

Live Load	4.8 kPa
Partition / mechanical Load	1.0 kPa
Total Specified Load	5.8 kPa

Note that the self-weight of the slab and the deck is NOT included in this calculation as the charts have subtracted this weight out when computing capacity.

Using the VicWest® HB308 tables with ZF75 Galvanizing, with 141mm slab thickness and 0.91mm deck, a 3200mm span with two or three spans continuous you get a maximum specified load of 8.8kPa, so OK. Note however that no load is given for the one-span condition. This is because the deck no longer works as a form when evaluated as a single span. Deck is capable of resisting the specified loads desired but should be two or three spans continuous to aid in its resistance to the weight of wet concrete.

The concrete on the floor deck will usually be reinforced to reduce cracking, but this reinforcement is not used to give the deck-slab a negative moment capacity. The capacity of composite deck is based only on the positive moment capacity and no increase is given for the capacity of the composite deck if it is continuous over two or more spans. Span tables include capacities for composite deck with one, two or three spans. The ability of the deck to span greater distances when acting as formwork increases when the deck is continuous over two or preferably three spans. Reinforcement is always required for fire-rated assemblies.

Concrete on metal deck systems, which act as a series of simple spans with no negative moment capacity, has significantly different structural behaviour to that of a concrete beam and slab system which is considered to be continuous. In the concrete system we consider compatibility torsion to be present in the edge beam and attribute additional load to the first interior beam. In the steel system there is no torsion on the edge beam and no increase in load for the first interior beam. A comparison between assumed behaviour of concrete slabs and concrete on a metal deck is shown in Figure 7.18.

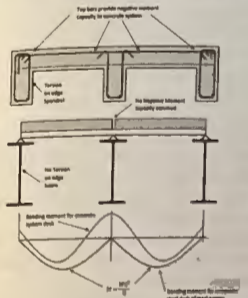


Figure 7.18. Comparison between the assumed behaviour of concrete slabs and concrete on metal deck

Care should be taken that the tables used for evaluating the composite deck capacity have the correct galvanizing finish used. The galvanizing can affect the shear bond and hence the composite deck capacity. For some spans and concrete thickness it is possible for a deck with ZF075 galvanizing to have almost 50% more live load capacity than a deck with Z275 galvanizing. ZF075 is referred to as "wipe-coat" or "galvannealed" and is often used where corrosion is not excessive. Z275 galvanizing is heavier and is used where more corrosion exists. The number after the Z refers to the total weight of galvanizing on both sides of the deck in g/m² of coating. The usual practice is to use ZF075 finish on floor deck in buildings

that are environmentally conditioned to human comfort where corrosion is not expected to be an issue. A comparison of the capacity of two deck slabs with similar deck section and topping thickness, but different weights of galvanizing, is shown in Figure 7.19.

Base Steel Nominal Thickness (mm)	Span (mm)	1 Span	2 Span	3 Span
0.91	2400	9.2	9.2	9.2
	2600	8.2	8.2	8.2
	2800	7.3	7.3	7.3
	3000	6.6	6.6	6.6
	3200		6.0	6.0
	3400			

HB 308 x 0.91mm deck with Z- 275 Galvanized deck finish (65mm concrete topping)

Base Steel Nominal Thickness (mm)	Span (mm)	1 Span	2 Span	3 Span
0.91	2400	14.6	14.6	14.6
	2600	12.7	12.7	12.7
	2800	11.1	11.1	11.1
	3000	9.9	9.9	9.9
	3200		8.8	8.6
	3400			

HB 308 x 0.91mm deck with ZF- 75 Galvanneal deck finish (65mm concrete topping)

Figure 7.19. Comparison of the composite deck capacity for composite deck with Z275 and ZF75 finishes. The increased thickness of galvanizing in the Z275 deck produces a lower shear bond between the concrete and the deck and the rated capacity of the deck is in some cases reduced by about 40%

Reinforcement for the concrete on the steel deck should satisfy A23.3 rules and provide 0.002 times the area of concrete in each direction using only the cross-sectional area of concrete above the flutes. No additional reinforcement is added for the concrete in the deck flutes as they are assumed to be reinforced by the deck steel. Reinforcement can be in the form of welded wire mesh or reinforcing bars. Reinforcing bars are easier to control in terms of getting the bars in the centre of the concrete. CSSBI recommendations for reinforcing are as shown in Table 7.14.

Table 7.14. CSSBI Recommendations for reinforcing

Topping thickness over flutes (h_c)	Area of reinforcing (mm^2/m)
(h_c) < 80mm	60
80mm < h_c < 150mm	$3h_c$-180
150mm < h_c	$1.8h_c$

Usually a designer will choose which type of deck is to be used (76mm x 0.91mm is common for floor systems on steel beams, while 38 x0.91 is common for floor systems supported on open web steel joists) and then determine the maximum span between supports such that shoring of the deck is not required during concrete placement. The composite deck floor tables tell the designer where shoring is required during concrete placement by not giving deck capacity values if shoring is required.

Tables given by the steel deck manufactures list "slab weight" and concrete volume. Back-calculating the weight of concrete using the standard weight shown in the deck manufacturer's table will usually show that the weight of the deck itself has been included in the slab weight.

SLAB THICKNESS h(mm)	101
Slab Weight, W_1(kN/m^2)	1.89
Concrete Volume, (m^3/m^2)	0.078

EXAMPLE—VICWEST® HB938 WITH 63MM TOPPING

Total deck height = 38mm + 63mm = 101mm.

Slab weight is given as 1.89kPa.

Concrete volume is given as 0.078m^3/m^2.

Concrete Weight = Concrete volume * concrete density of 2300kg/m^3

2300kg/m^3 = 22.6kN/m^3

Concrete Weight = 0.078m^3/m^2 * 22.6kN/m^3 = 1.76kPa

Listed Slab weight	1.89 kPa
Concrete Weight	1.76 kPa
Remaining Weight	0.13 kPa

0.13kPa = 13.3kg/m^2. This corresponds to HB938 x 1.22mm deck.

Volumes of concrete, and hence assumed weight of slab, can vary depending on what deck manufacturer has been selected by the contractor. Calculations should be based on a reasonable assumption of slab weight; this would mean the heaviest slab weight for the deck sections shown on structural drawings as being approved. Chien and Ritchie in *Design and Construction of Composite Floor Systems* also recommend that an increase in the concrete weight be allowed for the defection of the deck due to ponding under the wet weight of the concrete.

Slab thickness can vary from that desired if laser levels are used for setting slab elevation. If beams are cambered, the thickness of the slab may be less than desired if the camber does not come out under the weight of the wet concrete. If the beam deflects, additional concrete will be added to bring the level up, adding unexpected weight. It is best to have concrete thickness measured from the deck itself rather than an arbitrary reference point.

S16 Clause 17.1 requires that the minimum concrete thickness be 65mm "unless the adequacy of a lesser thickness has been established by appropriate tests." As a result of this onerous requirement, or possibly because of the difficulty of fitting reinforcing into a thinner slab, the cover slab thickness for composite decks is never less than 65mm. A23.3 Clause 21.10.7.2 requires that composite concrete toppings on steel deck may be used as a diaphragm only if they are at least 60mm thick. Steel deck composite tables almost always use 65mm as the minimum topping thickness. As a working surface for a seldom-accessed fan room, a 25mm topping may be used with deck, but this would not be considered a composite floor system and, in this case, the deck should be designed to support the weight of the topping as a dead load along with the live load. With a 25mm thick topping, the capacity of the deck is based on the deck acting alone, similar to roof deck.

All steel deck tables have warning messages against the use of deck with concentrated loads or moving loads. The Building Code has a requirement for concentrated loads that must be carried by the deck system and these requirements are often included in the calculation of deck tables published by the deck manufacturer. However, large moving point loads would include forklift or other similar construction equipment on the deck that can cause the concrete to delaminate from the steel deck and result in a radical reduction of capacity of the floor system. To address this issue, some of the deck manufacturers have procedures, some of which are published, for determining the resistance of a composite deck subjected to point loads.

Calling up composite steel floor deck requires several items to be shown on structural drawings including:

- The direction of the deck flutes.
- The depth of the deck (40mm Nominal or 75mm Nominal)

- The thickness or gauge of the deck. If gauge is used, the desired thickness should be called up after.
- That the deck is galvanized and to be composite-type.
- The thickness of the concrete topping.
- Where possible, deck elevations or the fact that the deck is sloped should also be shown on plan.
- The reinforcement for the concrete topping.
- Details such as the requirement of having three spans continuous, the type of deck fastening and the spacing of fasteners at deck side laps and to steel structure should be given in general notes, even if this information is provided in specifications. A typical detail is shown in Figure 7.20.

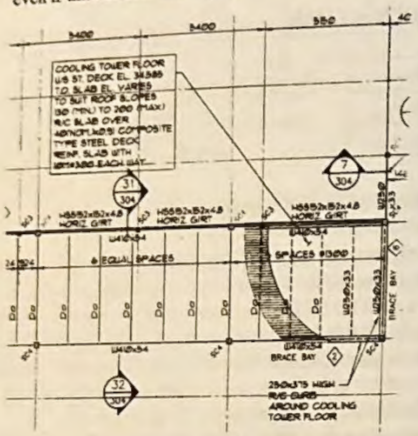


Figure 7.20. Calling up a composite floor deck on drawings. The direction of the flutes can be clearly seen along with the thickness of the deck, type of deck and the concrete thicknesses and reinforcing can be seen quickly

7.10 STEEL FLOOR DECK AS FORMWORK

In British Columbia, there is an unresolved debate between WorkSafeBC and the Canadian Institute of Steel Construction (CISC) over the appropriate loads to be used when designing deck as formwork. As a designer of floor deck you are in the middle of this debate. The position of WorkSafeBC is that steel floor deck is formwork and as such should be designed for the same loads that formwork is subject to. This means that the floor must be capable of carrying the weight of the concrete as a live load (Load Factor = 1.5) simultaneously with a live load of 2kPa (40psf) (also Load Factor = 1.5). CISC meanwhile feels that the wet concrete can be treated as a dead load (Load Factor = 1.25) and the live load simultaneous with this load is 1kPa (20psf). The steel side of the debate claims that WorkSafeBC is the only organization in North America that requires the 2kPa (40psf) live load on steel decks during concrete placement. The span tables produced by the deck manufacturer are generally based on the CISC assumptions, as shown in the excerpt from a composite deck chart set out as Figure 7.21. This means that at the top ends of the spans that do not require shoring it is possible to have a deck span that does not meet the requirements of WorkSafeBC while meeting the CISC requirements. The designer may wade into this debate, but it is worthwhile realizing the downside of producing a design that will require considerable extra cost and delay if it is necessary to shore the deck to meet WorkSafeBC requirements before placing concrete. The stretching of floor deck too far can make designs look very "lean," but can create issues at time of construction.

8. **Loads for the deck acting as a Form include Slab Weight, W_1 and a construction load of 1.0 kN/m² (21 psf) Uniformly Distrubuted Live Load OR 2.0 kN/m (137 lb/ft) Transverse Live Load.**

7.11 CONSTRUCTION "SITE OBSERVATION" OF FLOOR AND ROOF DECK

The "site observation" of steel deck consists of two parts: the review of shop drawings and a site review of the deck as installed. Review of shop drawings is necessary to verify if the assumptions of span continuity, deck profile and thickness are being met. Field review is performed to verify that the side-lap fastening and the fastening of the deck to supporting members has occurred at the spacing and locations requested. Care should be taken to verify that connections to members parallel to the deck span have been performed and that deck connections have occurred at the location of shear lugs and drag struts that are not part of the regular pattern of deck connections on the project. Puddle welds should be 19mm (3/4-inch) diameter with weld around the full perimeter of the hole, and there should be no relative movement between the supporting member and the deck when stepping near a puddle weld. Puddle welds require that the deck should be dry and good puddle welds are unlikely to be achieved if it is raining. If there is any question as to the effectiveness of the puddle welds, a testing agency should be consulted. The contractor will also have his or her own agency to do this work. If pins are used to fasten the deck to the supporting frame, the "set" of the pin should be measured and confirmed that they are within acceptable limits. Button punches should be inspected to see if there is a dimple present in both sheets. Floor deck is usually observed by the designer or his representative twice, once when the deck installation has been completed and once when the reinforcing steel or mesh is in place. Field review should also confirm that reinforcement has been placed around openings that disrupt any webs of the deck. When reviewing composite deck installation the deck should be checked to make certain it has the indentations necessary for composite action. If the deck is not composite, there are two choices:

1) Have the deck removed and replaced with composite deck.
2) Add reinforcing steel to the flutes of the deck to provide sufficient positive reinforcement for the concrete ignoring the deck steel. This will involve calculation on the part of the designer.

Composite deck should be clean and dry at the time of concrete placement. The designer should request the deck be cleaned – possibly by blowing off with high pressure air – if the deck surface is dusty, muddy, has standing water, has sawdust or other substances present that will disrupt the bond between concrete and steel deck.

7.12 STEEL SHEAR BOND CALCULATION

Load tables often give a series of numbers of either k_1 to k_4 or k_5 and k_6 that are used in computation of the shear bond capacity. These values are used in the shear bond equation giving V_r. An example of this might be as follows.

EXAMPLE—CALCULATE STEEL SHEAR BOND

These equations can be found in the CSSBI *Criteria for the Design of Composite Slabs*:

$$V_r = \phi_v * b * d\left[\frac{k_1 t}{l'} + \frac{k_2}{l'} + k_3 t + k_4\right]$$

An alternative equation is: $V_r = \phi_v * b * d\left[\frac{k_5}{l'} + k_6\right]$

where b = unit slab width = 1,000mm
d = distance from extreme concrete compression fibre to centroid of deck
d = 141mm – 48.0mm = 93mm
t = base steel nominal thickness, mm.
$\phi_v = 0.70$
l' = shear span = span / 4 for uniform load

Apply this to uniformly loaded deck with t = 0.91mm with 65mm topping and 3200mm span.

Apply the values given: $V_r = \phi_v * b * d\left[\frac{k_1 t}{l'} + \frac{k_2}{l'} + k_3 t + k_4\right]$

$\frac{k_1 t}{l'}$	$\frac{203.1197*0.91}{800mm}$	0.2310
$\frac{k_2}{l'}$	$\frac{2.3277}{800mm}$	0.0029
$k_3 t$	0.0510 * 0.91mm	0.0464
k_4	0.0456	0.0456
		Sum = 0.3260

$V_r = 0.7 * 1{,}000mm * 93mm * 0.3260 = 21.221\ N = 21.2kN$

Determine maximum factored load q_{max} including self-weight

$(3.2m / 2) * q_{max} = 21.2kN$

$q_{max} = 13.26kPa$

$$q_{service} = \frac{13.26kPa - 1.25*(2.14\,kPa)}{1.5} = 7.1kPa$$

7.13 COMPOSITE DECK MOMENT CAPACITY CALCULATION

Moment capacity of composite deck sections is computed assuming that the deck is acting as the reinforcing for the composite section, setting up a situation where the deck is in tension and the upper part of the concrete is in compression. The most typical case is that of a section below the balance point (that is, the quantity of steel is such that all steel can yield before crushing failure of the concrete occurs). This can be handled in the following manner:

Compute $c/d = \frac{A_s * F_y}{0.85 * f'_c * b * d * \beta_1}$

where: A_s = Area of steel in deck in 1,000mm width in mm^2
F_y = Yield stress of deck in MPa (usually 230MPa)
f'_c = Concrete strength in MPa.
b = 1,000mm
d = depth of steel centroid in composite section from top of slab
β_1 = Concrete coefficient = 0.81 for f'_c =25MPa

1) Compute c/d for balanced section and check that section of interest is below this value:

$$c/d(bal) = \frac{609*(h - d_d)}{(609 + F_y)*d}$$

where: h = height of composite section
d_d = depth of deck (76mm or 38mm)

2) If c/d for section is not below the balanced value, then the following steps are not valid. Refer to CSSBI information for determining the moment capacity of sections that are over-reinforced.

3) Compute the depth of compression block = $a = \frac{\phi_s * A_s * F_y}{0.85 * \phi_c * f'_c * b}$

Where ϕ_s = Capacity reduction factor for steel = 0.9. Note that the standard value of 0.85 for concrete reinforcement is not used in steel deck moment capacity calculations.

ϕ_c = Capacity reduction factor for concrete = 0.65

1) Moment resistance = $M_r = \phi_s * A_s * F_y * (d - \frac{a}{2})$

2) Compute the superimposed live load that can be supported as follows:

$M_r = \frac{W_f * L^2}{8}$

where W_f = Factored loading = (Live load *1.5) + (Dead load including self-weight * 1.25)

Note: The live load capacity controlled by moment is only one of several checks to be made. If the live load capacity due to moment is greater than the live load capacity determined from shear bond calculations, then the shear bond calculation will govern the live load capacity.

EXAMPLE—CAPACITY OF COMPOSITE DECK FROM FIRST PRINCIPLES

Compute the moment capacity and live load capacity of a composite deck section having the following properties: $A_s = 1513mm^2$, $f'_c = 25MPa$, 65mm topping on 76 x 0.91 deck giving overall height of 141mm, d = 93.0mm.

$$c/d = \frac{A_s * F_y}{0.85 * f'_c * b * d * \beta_1} = \frac{1513mm^2 * 230MPa}{0.85 * 25\,MPa * 1000mm * 93.0mm * 0.85} = 0.21$$

$$c/d(bal) = \frac{609*(h - d_d)}{(609 + F_y)*d} = \frac{609*(141mm - 76mm)}{(609 + 230\,MPa)*(93mm)} = 0.507$$

OK, as c/d < c/d (bal) Section is not over-reinforced and remainder of sections apply.

$$a = \frac{\phi_s * A_s * F_y}{0.85 * \phi_c * f'_c * b} = \frac{0.9 * 1513mm * 230MPa}{0.85 * 0.65 * 25MPa * 1000mm} = 24.6mm$$

$M_r = \phi_s * A_s * F_y * (d - \frac{a}{2})$

$M_r = 0.9 * 1513mm^2 * 230{,}000\frac{kN}{m^2} * 93mm - \frac{24.6mm}{2} * 10^{-9} mm^3/m^3$

$M_r = 25.27kNm$

Determine maximum superimposed live load that can be supported by the deck if span is 3200mm

$M_r = \frac{W_f * L^2}{8} = 25.27\ kNm = \frac{W_f * 3.2^2}{8}$

Solve for $W_f = \frac{25.27kNm * 8}{(3.2m)^2} = 19.74kN/m$

If slab weight is 2.14kN /m^2, LL = 11.4kPa

Note that this is considerably greater than the LL found by shear bond analysis and therefore in this case the bending moment is not the governing criterion.

7.14 COMSLAB® DESIGN

ComSlab®, manufactured by Bailey Metal Profiles, is a relatively new type of floor deck that has an 8 inch (208mm) depth. The product was previously known as ComFloor. The deck section is used only in conjunction with concrete as a composite system; it is never used without concrete as a roof deck. With a cover slab of 65mm, the depth of the section can be as thin as 265mm and at that depth can easily span

8m if shoring is used during concrete placement, and spans of up to 4.5m can be achieved without shoring during concrete placement. This makes the product well suited to clear spanning between support lines in the walls of the building and the product has been used in hotels, prisons and schools. By clear-spanning between the wall lines, the structural depth of the framing system over the room can be less than the structural depth required with traditional metal deck on steel beam systems. For example, a 265mm deep ComSlab® system can easily span 7.4m supporting a partition load of 1kPa with a live load of 2.4kPa for a total service load of 3.4kPa. Using standard steel framing with 65mm topping on 38 deck on W360 beams at 1500 o/c gives a total structural depth of 463mm. In this case, the use of the ComSlab® system saves 200mm of structural depth and provides a ceiling without beam lines.

For a deck thickness of 0.91mm, ComSlab® spans greater than approximately 3.5m need shoring and the issue of WorkSafeBC formwork loads may also be of concern. The ComSlab® catalogue gives two values of maximum unshored span: (1) maximum unshored span for a live load of 1kPa, and (2) maximum unshored span for a live load of 2kPa. For projects in British Columbia, the 2kPa construction live load should be used.

The reinforcing bars that are placed in each deck flute provide much of the moment resistance of ComSlab® decks. ComSlab® tables are used to determine the bar size required to match the span and loading conditions.

In choosing a ComSlab® system the designer should show the following information:

- The layout and extent of the ComSlab® giving the direction of the webs
- The thickness of the deck (this is restricted to a choice of 0.91mm or 1.22mm, as ComSlab® does not have 0.76mm or 1.52mm)
- The thickness and concrete properties of the topping
- The reinforcing in the flutes
- Any reinforcing or special details required for the deck, including extra reinforcement to tie diaphragms to shear resisting elements
- Details related to support of the deck at span ends
- Appropriate notes including a flag when shoring is required.

EXAMPLE—COMSLAB® EXPRESSED ON STRUCTURAL DRAWINGS

The information shown on drawings is similar to that for standard composite deck systems. The only difference is that with ComSlab®, the reinforcing in the deck flutes must also be called up as this is selected by the designer. The example shown in Figure 7.21 still refers to ComFloor, but current drawings should refer to ComSlab®.

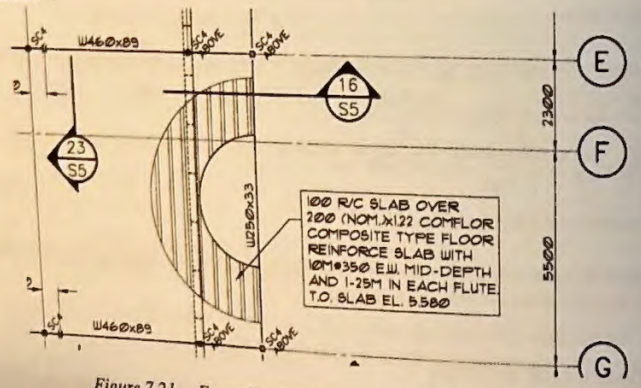


Figure 7.21. Example of calling up ComSlab® on drawings. The direction of the flutes, the reinforcing steel in each flute and the reinforcing of the slab can be seen. The elevation of the top of the slab is called up

Further information on ComSlab® can be found in the literature produced by Bailey Metal Profiles. The use of the product is becoming more common in Western Canada and contractors are becoming more familiar with the system.

Care should be taken when designing ComSlab® to locate openings away from deck flutes, as special procedures and support, possibly including steel beams, will be required at locations where the deck flute load must be picked up.

ComSlab® with a 64mm cover slab will have a one-hour fire rating, while ComSlab® with a 90mm cover slab will have a one-and-one-half-hour fire rating. If you are attempting to use ComSlab® for a two-hour fire rating it may be necessary to add additional fire proofing below. Research is ongoing in this area and you should check with the deck manufacturer for updates.

7.15 "OTHER" SYSTEMS FOR THE MATERIAL SPANNING BETWEEN BEAMS

The "plate" that sits on top of the steel beams is not always a roof deck system or a composite concrete floor system. The following four options are occasionally encountered in typical building design and some are very common in industrial structures.

1) **Glass:** Structural steel supporting glass is most often encountered in canopy design and support of window walls. While the design of the glass is usually by a speciality engineer, the design of the back-up system will usually be designed by the building engineer. The glass should not be considered as a diaphragm for restraining the structural steel.
2) **Checker Plate:** Checker plate is formed by adding indentations to steel plate to provide a surface that is less slippery than an undeformed plate. Typically checker plate is encountered in non-public stair landings and catwalks, although it may be encountered in stairs where the architect has desires to be "edgy." Checker plate is typically in 6mm thickness, although greater thickness can be special-ordered. The feel of a checker plate area will be "lively."
3) **Safety Grip Channel:** Safety grip channel is a manufactured system that is occasionally used for longer catwalks or emergency exits. It comes in thickness of 2 inches and 3 inches and is typically galvanized. The channels are clipped to the supporting structural steel and should not be considered to be a diaphragm when considering the unsupported length of the supporting members. Structural drawings should clearly show the size, gauge, and orientation of the safety grip channel sections. When used as a work platform or catwalk where there are workers below, there should be a 6mm thick by 125mm high "kick plate" at the edges to avoid tools and pieces from becoming falling hazards. A guardrail will also be required at edges.
4) **Metal Grating:** Metal grating is also a manufactured system that is occasionally used for longer catwalks or emergency exits and may be used as a work platform. It comes in thickness of 1-1/2 inches and may be galvanized. The metal grating is clipped to the supporting structural steel and should not be considered to be a diaphragm when considering the unsupported length of the supporting members. When used as a work platform or catwalk where there are workers below there should be a 6mm thick by 125mm high "kick plate" at the edges to avoid tools and pieces from becoming falling hazards. A guardrail will also be required at edges.

CHAPTER 8
THE ART AND SCIENCE OF FRAMING

The real art in structural steel design is the conceptual layout of the framing system. The laying out of beams and girders to achieve an economical and efficient system is an important first step in any design. While many problems will break down to laying out beams in rectangular bays, special problems involving cantilevers, curved edges and awkwardly shaped areas are all too often part of the design process. Vertical holes for ducts, elevators, and stairs must be accommodated in the framing grid. Ducts and conduits often pierce beam webs and may affect the size and type of beams selected for the floor system. This chapter will review some of these framing issues and the challenges that arise. In addition, we will review and try to simplify some of the Canadian snow drifting requirements for the frequently encountered case of a roof step or regions of a roof located near a mechanical penthouse, and we will discuss the effect that this snow drifting can have on the layout of roof beams.

8.1 GUIDELINES FOR LAYOUT OF STEEL FRAMING

Many framed areas are rectangular with the repetitive beams running perpendicular to the girders. The designer lays out the beams to give even spacing between the columns, keeping in mind not to exceed the maximum span of the deck. While variations exist, depending on designer preference, deck capacity and loading conditions, a starting point for typical spans of floor and roof deck in non-drift areas would be as shown in Table 8.1.

Table 8.1. Deck capacity and loading conditions

	Roof Deck	Floor Deck
38mm deck	2.0m	1.5m
76mm deck	2.5m	2.5m

For most suspended floors and roofs there is a rectangular grid of columns that provides a load path to take the loads from the floor and roof framing to the foundation level. The grid of columns means that it is necessary to design beams and girders to frame in two directions, as shown in Figure 8.1, which shows a single bay of framing.

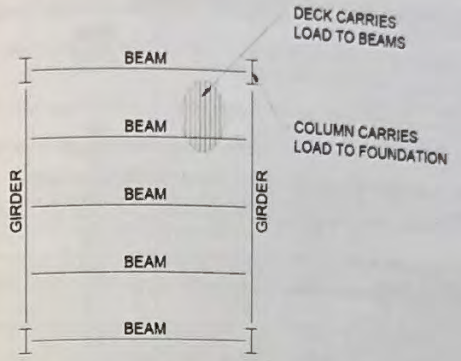


Figure 8.1. Single bay of framing

The beams and girders are linear elements and the beams must be picked up by the girders, which in a rectangular layout run perpendicular to the span of the beam and parallel to the span of the deck.

For stability, ease of erection and plumbing of columns, it is preferable if columns have beams framing into them from two directions. Adjust framing of each bay such that there is a beam on each column line. This places the load of at least one beam line directly into the column without having to be carried by a girder and provides a direct brace to the column. Unless there are overriding circumstances, such as snow drifting or floor openings, the beams are spaced evenly along the supporting girders. Figure 8.2 shows two unequal bays; the framing on the left with a beam on the column line and each bay equally spaced is the preferred option.

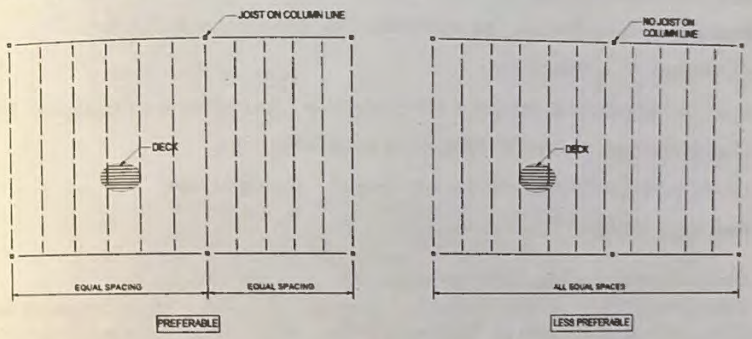


Figure 8.2. Options for beam layout with two unequal bays of framing

Selection of column spacing will often be determined by the functional requirements of the space where the columns sit. In buildings with defined rooms such as schools or hospitals, columns are located in walls so they don't hamper activities occurring in the rooms below. In buildings such as big box stores and warehouses, which have large open spaces where columns can be tolerated, the column spacing will normally be chosen to be uniform. As there is a relationship between the span of the beam and its depth, the spacing of columns may be limited by the available structural depth. The weight of structural steel increases as the span increases, which will also have an influence on column spacing. Assembly occupancy loading of a hotel lobby is shown in Table 8.2.

Table 8.2. Hotel lobby loading (assembly occupancy)

Load Source	Dead (kPa)	Live (kPa)	Factored (kPa)
Hotel Lobby loading (Code Table 4.1.5.3)		4.8	7.2
Partition load / finishes / suspended mech. & elect.	1.0		1.3
65 topping on 75 deck	2.1		2.6
Total load for Deck Design	**3.1**	**4.8**	**11.1**
Floor Beams	0.3		0.4
Total load for Beam Design	**3.3**	**4.8**	**11.5**
Girders	0.1		0.1
Total load for Girder Design	**3.4**	**4.8**	**11.6**

EXAMPLE—DESIGN BEAMS AND GIRDERS FOR A FLOOR BAY

Design the beams and girders for a 7.5m x 7.5m floor bay to carry the load of a hotel lobby. Assume similar sizes in adjacent bays. Beams are to be spaced at 2.5m o/c. Deck is 76mm with 65mm topping. There are no architectural restrictions as to beam depth.

Solution:
Design beams and girders and compute weight.

Beam design: Span = 7.5m, Tributary width = 2.5m

$M_f = wL^2/8 = 2.5m * 11.5kN/m^2 * (7.5m)^2 / 8 = 202kNm$

From beam selection tables, the least weight beam that satisfies this is W410x39

Try W410x39, $M_r = 227kNm$

$w_s = 2.5m * 4.8kN/m^2 = 12.0kN/m$ $I_x = 126x10^6mm^4$ L = 7.5m Deflection = 19.6mm = L / 382

Check span to depth: 7500/410 = 18.3 (OK – should be close to 20)

Use W410x39 $M_r = 227kNm$ Deflection = 19.6mm = L / 382 (OK<L/360)

Beam weight = 39kg/m / 2.5m = $15.6kg/m^2$

Girder design: Span = 7.5m, Tributary width 7.5m

(The tributary width is 7.5m as we assume there is an adjacent matching 7.5m x 7.5m panel that also contributes load to the girder, so we get 7.5m/2 tributary from one side of the girder and 7.5m/2 tributary from the other side.)

(Note that 7.5m x 7.5m = $56m^2$. This is less than the $80m^2$ required for live load reduction for assembly occupancy loading.)

$M_f = wL^2 / 8 = 7.5m*11.6kN/m^2 * (7.5m)^2 / 8 = 611kNm$

From beam selection tables, least weight beam that satisfies this is W610x82

Try W610x82, $M_r = 683kNm$

$w = 7.5m*4.8kN/m2 = 36kN/m$ $I = 560x106mm^4$ L = 7.5m Deflection = 13.2mm = L / 566 OK

Check span to depth: 7500/610 = 12.2 (the beam is much deeper than span / 20 however would expect heavily loaded girders to be more stocky than typical beams)

Use W610x82 Girder

Girder weight = 82kg/m / 7.5m = $10.9kg/m^2$

Total weight of structural steel (not including columns) = $15.6kg/m^2 + 10.9kg/m^2$ = $26.5kg/m^2$

There will be less structural steel weight in the floor if the spacing between columns is kept down. Repeating the analysis from the previous exercise using a column spacing that produces bays of 10m x 10m and 12.5m x 12.5m, we find that the weight of the floor members is lower when the grid spacing is lower. This is illustrated in the graph and chart shown in Figure 8.3 and Figure 8.4.

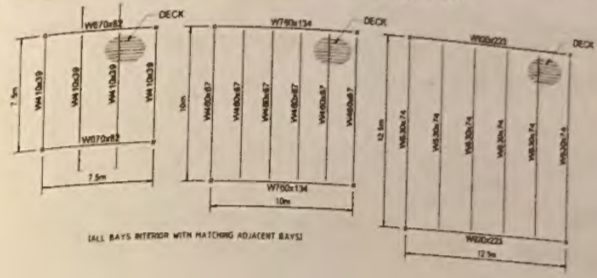


Figure 8.3. Weight of floor members with various grid spacings

	SQUARE BAY SIZE (M)		
	7.5	10	12.5
Beam Weight (kg / sq m)	15.6	26.3	29.6
Girder Weight (kg / sq m)	10.9	13.4	17.8
Total Weight (kg / sq m)	**26.5**	**39.7**	**47.4**

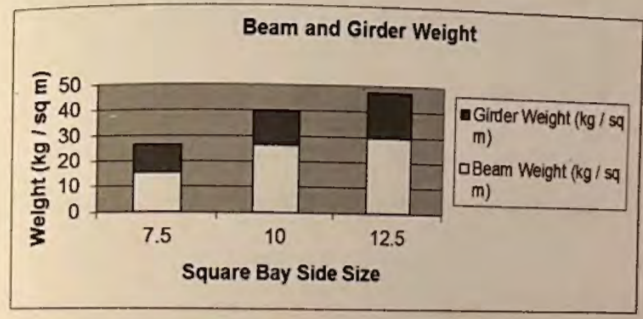


Figure 8.4. Beam and girder weight compared with square bay side size

It might be argued that the economy of the steel is not just a matter of steel weight and that there is a saving from using the reduced number of members when larger bays are used, such as in the reduced number of columns and footings required. However, the almost doubling of the beam and girder weight per square metre when going from a 7.5m x 7.5m to 12.5m x 12.5m bay will have a significant influence on cost. As structural engineers, we usually have other factors to deal with, including the conflicting aims of maximum structural depths and minimum column spacing.

For framing bays that are rectangular but not square, the framing will generally be better if the repetitive element runs in the short direction. This is illustrated in the following example. Caution: This rule

is reversed when open web steel joists (OWSJs) are used. We will review this further in the section on OWSJs.

EXAMPLE—DESIGN THE BEAMS AND GIRDERS FOR A HOTEL LOBBY

Using the data in Table 8.3 and the beam span shown in Figure 8.5, design the beams and girders for a 7.5m x 10m floor bay to carry the load of a hotel lobby. Assume similar sizes in adjacent bays. Beams are to be spaced at 2.5m o/c. Deck is 76mm with 65mm topping. There are no architectural restrictions as to beam depth.

Table 8.3. Hotel lobby loading (assembly occupancy)

HOTEL LOBBY LOADING (ASSEMBLY OCCUPANCY)			
Load Source	**Dead (kPa)**	**Live (kPa)**	**Factored (kPa)**
Hotel Lobby loading (Code Table 4.1.5.3)		4.8	7.2
Partition load / finishes / suspended mech. & elect.	1.0		1.3
65 topping on 75 deck	2.1		2.6
Total load for Deck Design	**3.1**	**4.8**	**11.1**
Floor Beams	0.3		0.4
Total load for Beam Design	**3.3**	**4.8**	**11.5**
Girders	0.1		0.1
Total load for Girder Design	**3.4**	**4.8**	**11.6**

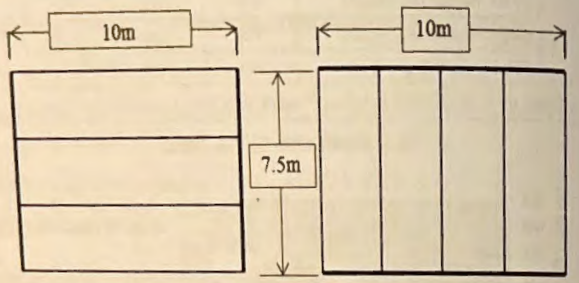


Figure 8.5. Beam span

a) Beams span in long direction:

Beam design: Span = 10m Tributary width = 2.5m

$M_f = wL^2/8 = 2.5m * 11.3kN/m^2 * (10m)^2 / 8 = 356kNm$

From beam selection table, least weight beam that satisfies this is W460x60.

Try W460x60 $M_r = 397kNm$

$w_s = 2.5m * 4.8kN/m^2 = 12.0kN/m$ $I = 255x10^6mm^4$ $L = 10m$ Deflection = 30.6mm = L / 326

Need $I = (360 / 326)*255 x10^6mm^4 = 281 x10^6mm^4$

Use W530x66 $M_r = 484kNm$ Deflection = 22.3mm = L / 449

Check span to depth: 10,000/530 = 18.8 (OK – should be close to 20)

Beam weight per bay = 3 * 10m * 66kg/m = 1980kg

Girder design: Span = 7.5m, Tributary width 10m

(The tributary width is 10m as we assume there is an adjacent matching 7.5m x 10m panel that also contributes load to the girder so we get 10m/2 tributary from one side of the girder and 10m/2 tributary from the other side.)

(Note that 7.5m x 10m = $75m^2$. This is less than the $80m^2$ required for live load reduction for assembly occupancy loading.)

$M_f = wL^2/8 = 10m*11.5kN/m^2 * (7.5m)^2 / 8 = 809kNm$

From beam selection table, least weight beam that satisfies this is W610x101.

Try W610x101 $M_r = 900kNm$

$w = 10m*4.8kN/m^2 = 48kN/m$ $I = 764x10^6mm^4$ $L = 7.5m$ Deflection = 12.9mm = L / 579 OK

Check span to depth: 7500/610 = 12.2 (the beam is much deeper than span / 20 however would expect heavily loaded girders to be more stocky than typical beams)

Use W610x101 Girder

Girder weight per bay = 1 * 7.5m * 101kg/m = 758kg

b) Beams span short direction:

Beam design: Span = 7.5m, Tributary width = 2.5m

$M_f = wL^2/8 = 2.5m * 11.3kN/m^2 * (7.5m)^2 / 8 = 199kNm$

From beam selection table, least weight beam that satisfies this is W410x39.

Try W410x39 $M_r = 227kNm$

$w_s = 2.5m * 4.8kN/m^2 = 12.0kN/m$ $I = 126x10^6mm^4$ $L = 7.5m$

Deflection = 19.6mm = L / 382 OK

Check span to depth: 7500/410 = 18.3 (OK – should be close to 20)

Use W410x39 beam

Beam weight per bay = 4 * 7.5m * 39kg/m = 1170kg

Girder design: Span = 10m Tributary width 7.5m

$M_f = wL^2/8 = 7.5m*11.5kN/m^2 * (10m)^2 / 8 = 1078kNm$

From beam selection table, least weight beam that satisfies this is W610x125 (it might be argued that we should use the W690x125 that is highlighted but will save the 70mm of beam depth as the W610x125 and W690x125 are the same weight).

Try W610x125 $M_r = 1250kNm$.

$w = 7.5m*4.8kN/m^2 = 4.8kN/m$ $I = 985x10^6mm^4$ $L = 10m$ Deflection = 23.8mm = L / 420 OK

Check span to depth: 10,000/610 = 16.4 (OK – a bit stocky – typical beam would be span / 20 but would expect that with a loaded girder the most efficient size would be deeper than L/20)
Use W610x125 Girder

Girder weight per bay = 1 * 10m * 125kg/m = 1250kg

This is illustrated in Figure 8.6 and Figure 8.7 and Table 8.4, Table 8.5, and Table 8.6.

Results:

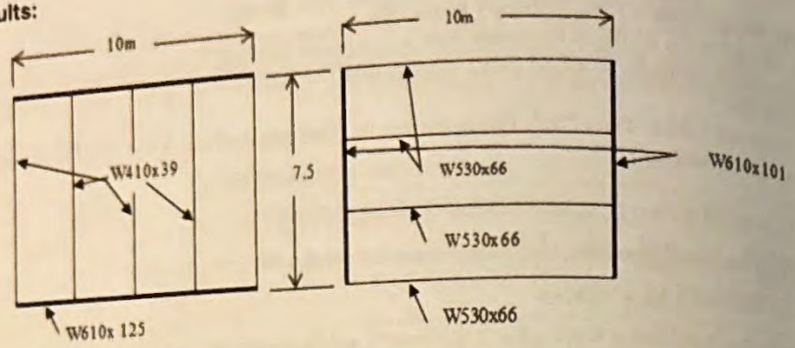


Figure 8.6. Girder design

Table 8.4. Beam weight

	Beam Weight	Girder Weight	Total Weight
Beams span long direction.	1980 kg	758 kg	2738 kg
Beams span short direction.	1170 kg	1250 kg	2420 kg

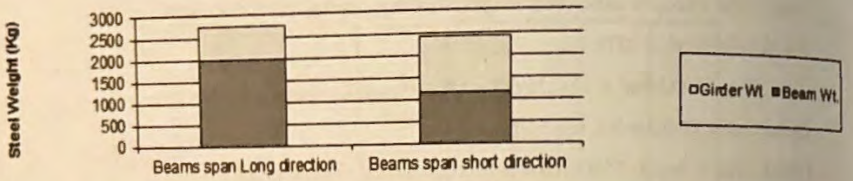


Figure 8.7. We save 12% of the steel weight by having the beams span the short direction

Table 8.5. Panel deflection

	Beam Deflection	Girder Deflection	Total Deflection
Beams span long direction.	30.6mm	12.9mm	43.5mm
Beams span short direction.	19.6mm	23.8mm	43.4mm

Table 8.6. Beam depth

	Beam Depth	Girder Depth
Beams span long direction.	530mm	610mm
Beams span short direction.	410mm	610mm

8.2 JOIST FRAMING ISSUES

In the previous section, it was concluded that when framing bays that are rectangular, but not square, the framing will generally be better if the repetitive element runs in the short direction. It was cautioned that this may not be true when OWSJs are used. Repeating the previous example, but using OWSJs, this point can be illustrated.

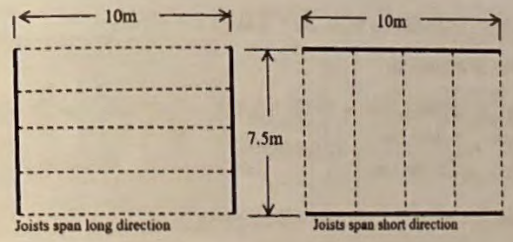
In the example that follows, the joists are supported on girders and the weight saving comes from reducing the weight of the girder. If the joists were to be supported on masonry or tilt-up walls, the more economical solution would be to have the joists span the short direction.

EXAMPLE—DESIGN OWSJS AND GIRDERS

Using data in Table 8.7 and the joists span illustrated in Figure 8.8, design the OWSJs and girders for a 7.5m x 10m floor panel to carry the load of a hotel lobby. Assume similar sized adjacent panels. OWSJs are to be spaced at 2.5m o/c. Deck is 38mm with 65mm topping. There are no architectural restrictions as to girder or joist depth.

Table 8.7. Hotel lobby loading (assembly occupancy)

Load Source	Dead (kPa)	Live (kPa)	Factored (kPa)
Hotel Lobby loading (Code Table 4.1.5.3)		4.8	7.2
Partition load / finishes / suspended mech. & elect.	1.0		1.3
65 topping on 75 deck	2.1		2.6
Total load for Deck Design	**3.1**	**4.8**	**11.1**
Floor Purlins (joists)	0.2		0.3
Total load for Purlin Design	**3.3**	**4.8**	**11.3**
Girders	0.1		0.1
Total load for Girder Design	**3.4**	**4.8**	**11.5**



a) *b)*

Figure 8.8. Joists span: a) long direction and b) short direction

a) Joists span in long direction:

Joist design: Span = 10m, Tributary width = 2.5m

$w_f = 2.5m * (11.3kPa) = 28.3kN/m$

A review of the tables shows that they do not run to this high a capacity. Add one more joist to reduce the tributary width to 7.5m / 4 = 1.875m.

$w_f = 1.875m * (11.3kPa) = 21.1kN/m$

Using the tables, determine economical joist is 800mm deep at 26.0kg/m

$w_s = 1.875m * 4.8kN/m^2 = 9.0kN/m$

Maximum service load for L/360 = 94% * (14kN/m) = 13.2kN/m OK (>9.0kN/m) Deflection =

$$\frac{10000mm}{360} * \frac{9kN/m}{13.2kN/m} = 18.9mm = \frac{Span}{528}$$

Use 800 Deep OWSJ at 26.0kg/m

Joist weight per bay = 4 * 10m * 21.1kg/m = 844kg

Girder design: Span = 7.5m, Tributary width 10m (Identical to previous example)

(Note that 7.5m x 10m = 75m². This is less than the 80m² required for live load reduction for assembly occupancy loading.)

$M_f = wL^2 / 8 = 10m*11.4kN/m^2 * (7.5m)^2 / 8 = 801kNm$

Span / 20 for floor beam = 7500/20 = 375, trying W410 would be minimum but would want girder depth to at least match the beam depth (minimum depth = 530mm).

Try W610x101 M_r = 900kNm

$w = 10m * 4.8kN/m^2 = 4.8kN/m$ $I = 764 \times 10^6 mm^4$ L = 7.5m Deflection = 12.9mm = L / 579 OK

Use W610x101 Girder

Girder weight per bay = 1 * 7.5m * 101kg/m = 758kg

b) Joists Span short direction:

Joist design: Span = 7.5m, Tributary width = 2.5m

7.5m joists not listed use 8m tables

For reasons listed, when evaluating joists spanning in long direction add one more joist per bay giving a tributary width of 10m / 5 joists = 2m / joist

$w_f = 2.0m * 11.4kN/m^2 = 22.6kNm$ $w_s = 2m * 4.8kPa = 9.6kN/m$

Try 700 Deep OWSJ at 22.0kg/m

Maximum Live load to satisfy L/360 = 107%*15.0kN/m = 16.05kN/m OK (>9.6)

Deflection = $\frac{7500mm}{360} * \frac{9.6kN/m}{16.05kN/m} = 12.5mm = \frac{Span}{602}$

Joists weight per bay = 5 * 7.5m * 22kg/m = 825kg

Girder design: Span = 10m, Tributary width 7.5m (Identical to previous example)

$M_f = wL^2/8 = 7.5m*11.4kN/m^2 * (10m)^2 / 8 = 1067kNm$

Try W610x125, M_r = 1250kNm. (The W690x125 girder is located at the top of the block in the beam selection tables but try the W610 beam as the beam weight is the same and the overall structural depth will be less.)

$w = 7.5m*4.8kN/m^2 = 4.8kN/m$ $I = 985x10^6 mm^4$ L = 10m Deflection = 23.8mm = L / 420 OK

Use W610x125 Girder

Girder weight per bay = 1 * 10m * 125kg/m = 1250kg

Joist and beam arrangement is illustrated in Figure 8.9. Beam weight is as shown in Table 8.8.

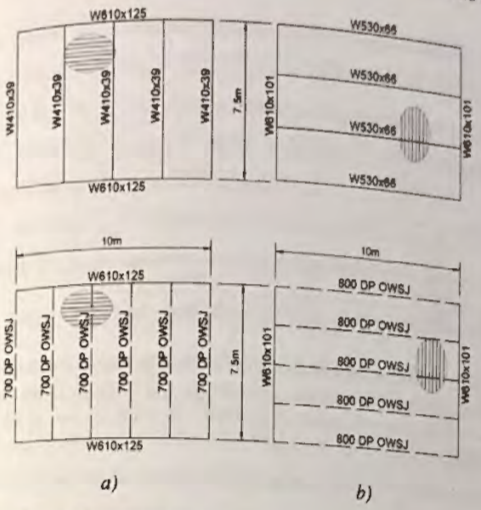


Figure 8.9. Joist and beam arrangement in a rectangular bay with repetitive elements running: a) in short direction and b) in long direction.

Table 8.8. Beam weight

	Purlin or Joist Weight	Girder Weight	Total Weight
Wide-flange beams span long direction	1980 kg	758 kg	2738 kg
Wide-flange beams span short direction	1170 kg	1250 kg	2420 kg
Joists span long direction	844 kg	758 kg	1602 kg
Joists span short direction	825 kg	1250 kg	2075 kg

As shown in Table 8.8, from a steel weight standpoint, the most economical solution is the joists spanning the long direction. This comparison can be expressed best in the bar chart shown in Figure 8.10.

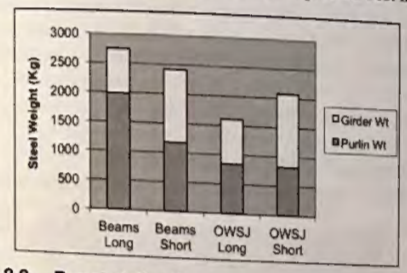


Figure 8.10. Comparison of beam weights and lengths when joists or beams are used in rectangular bays

Table 8.9. Panel deflection

	Purlin or joist Deflection	Girder Deflection	Total Deflection
Wide-flange beams span long direction	30.6mm	12.9mm	43.5mm
Wide-flange beams span short direction	19.6mm	23.8mm	43.4mm
Joists span long direction	18.9mm	12.9mm	31.8mm
Joists span short direction	12.5mm	23.8mm	36.3mm

As shown in Table 8.9, there is essentially no change in deflection at bay centrelines when using rolled sections. Framing the bay with joists results in lower deflections in this case.

Table 8.10. Beam depth resulting from four framing schemes

	Beam or Joist Depth	Girder Depth
Wide-flange beams span long direction	530mm	610mm
Wide-flange beams span short direction	410mm	610mm
Joists span long direction	800mm	610mm
Joists span short direction	700mm	610mm

As shown in Table 8.10, there is more space under the beams when they span the short direction. Joists are 50% to 70% deeper than the rolled sections. For example, consider the two 10.5m x 10.5m bays in Figure 8.11. The one on the left uses 38mm deck with beams at 1500mm o/c, while the one on the right uses 76mm deck with beams at 2625mm o/c.

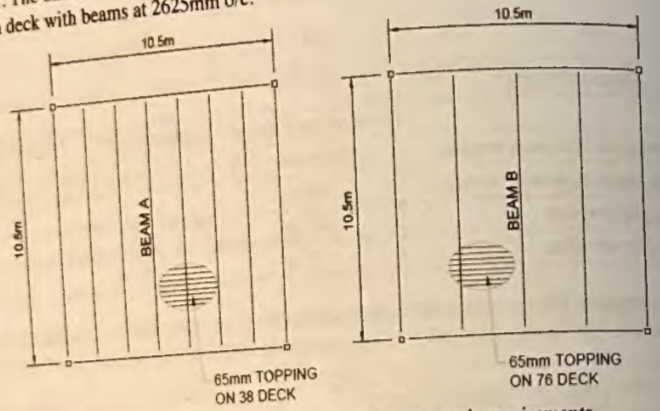


Figure 8.11. Beam design governed by strength requirements

The loading for these two systems is almost identical, as shown in Table 8.11.

Table 8.11. The loading for two systems shown to be nearly identical

Loading With 65mm on 38 deck			
Load Source	Dead (kPa)	Live (kPa)	Factored (kPa)
Assembly loading (Code Table 4.1.5.3)		4.8	7.20
Partition load / finishes / suspended mec	1		1.25
65 topping on 38 deck	1.89		2.36
Total load for Deck Design	**2.89**	**4.8**	**10.81**
Beams	0.35		0.44
Total load for Beam Design	**3.24**	**4.8**	**11.25**
Girders in either option (estimate)	0.15		0.19
Total load for Girder Design	**3.39**	**4.8**	**11.44**

Loading With 65mm on 76 deck			
Load Source	Dead (kPa)	Live (kPa)	Factored (kPa)
Assembly loading (Code Table 4.1.5.3)		4.8	7.20
Partition load / finishes / suspended mec	1		1.25
65 topping on 38 deck	2.14		2.68
Total load for Deck Design	**3.14**	**4.8**	**11.13**
Beams	0.3		0.38
Total load for Beam Design	**3.44**	**4.8**	**11.50**
Girders in either option (estimate)	0.15		0.19
Total load for Girder Design	**3.59**	**4.8**	**11.69**

Beam design with 38mm deck: Span = 10.5m, Tributary width = 1.5m

$M_f = wL^2/8 = 1.5m * 11.25kN/m^2 * (10.5m)^2 / 8 = 233kNm$

Try W360x45 $M_r = 242kNm$

$w_s = 1.5m * 4.8kN/m^2 = 7.2kN/m$ $I = 122x10^6mm^4$ $L = 10.5m$

Deflection = 46.7mm = L / 225 NO GO – does not meet L/360 criteria

Need $I = (360 / 225) * (122 \times 10^6mm^4) = 195 \times 10^6mm^4$

Try W460x52 $I = 212 x10^6mm^4$ $M_r = 338kNm$ (Span / Depth = 10.5/.460 = 22.8)

Use W460x52 beam at 1500mm o/c.

(Beam design is governed by deflection requirements).

Beam weight per square metre = 52.0kg/m / 1.5m = 34.6kg/m²

Beam design with 76mm deck: Span = 10.5m, Tributary width = 2.625m

$M_f = wL^2/8 = 2.625 * 11.50kN/m^2 * (10.5m)^2 / 8 = 416kNm$

Try W530x66 $M_r = 484kNm$

$w_s = 2.625m * 4.8kN/m^2 = 12.6kN/m$ $I = 351x10^6mm^4$ $L = 10.5m$

Deflection = 28.4mm = L / 370 OK (meets deflection criteria of L/360)

Use W530x66 beam at 2.625m o/c.

(Beam design is governed by strength requirements).

Beam weight per square metre = 65.7kg/m / 2.625m = 25.0kg/m²

The saving is $34.6kg/m^2 - 25.0kg/m^2 = 9.6kg/m^2$, a saving of 28% in terms of beam weight, but there is also a saving of three beams for each bay that need to be detailed, fabricated, shipped and erected.

In laying out the beams it is fundamental to remember that steel decking is a one-way system. The framing system must account for this by having supports for the deck at all deck flute ends. Designers who are well versed in the design of concrete floor systems that often span in two directions may find challenges when laying out steel framing systems requiring the placing of beams at all edges. A well-designed steel framing system will have beams framing around openings for ducts and other vertical penetrations and require beams or angles that act as beams at ends of all cantilevers.

In addition to thinking about the economy of the beam system, the designer must also consider the economy of the decking system. It is desirable to keep the deck flutes running in the same direction, avoiding unnecessary changes in deck direction as shown in Figure 8.12. Changes in deck direction are unavoidable on circular shapes and hip roofs, but changes in deck direction require deck supports at the location where the direction changes, and this will add framing cost and complexity. Changes in deck direction in floor decks will also require cell closures to hold the concrete in place.



Figure 8.12. Change in direction of an 8-inch deck section. Changing direction involves more cutting and fitting of the deck and there must be closure pieces placed in the concrete flutes to prevent concrete flowing under the deck

Most successful floor framing designs will attempt to frame in ways that reduce the number of moment connections, especially field welded moment connections. While moment connections may be required in frames used for lateral force resistance, it is desirable to try to reduce the number of moment connections required at cantilevers and in typical framing conditions. An example of treating a cantilever at the edge of a building is shown in Figure 8.13. If headroom conditions permit, it is preferable to place the supporting member under the cantilevering beams rather than trying to moment connect through the supporting beam.

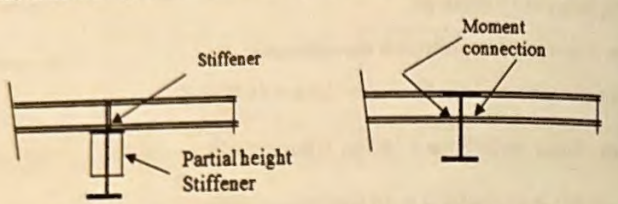


Figure 8.13. Moment connections in typical cantilever framing conditions

For roof overhangs it is often possible to avoid both moment connections and the depressing of a beam on the column line (Figure 8.13) by passing a beam over a column that acts as an outrigger to support a beam located on the outside edge of the roof. This is shown in Figure 8.14.

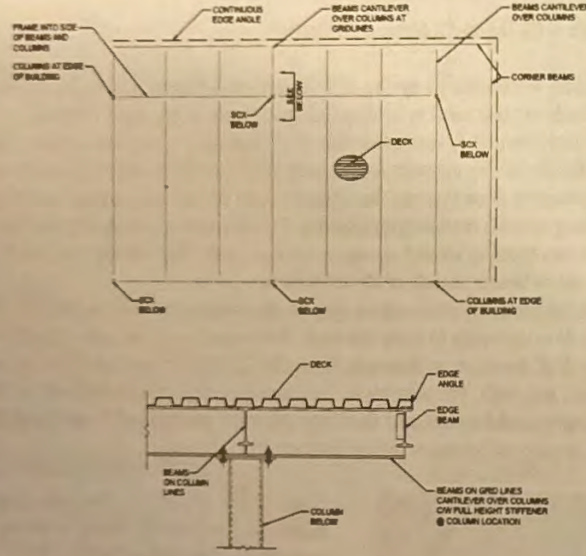


Figure 8.14. Roof framing that allows for a cantilevered eyebrow while eliminating the need for moment connections or a depressed beam. The corner also does not need any moment connections and the corner beams are longer than other beams in the system to avoid a moment connection at the corner. Eyebrows on roofs are increasingly popular as they allow for sun shading and protect the wall from some of the effects of rain

Where beams are not perpendicular to the supporting girders, the designer should avoid having beam intersections where the angle between the beam and the girder is less than 45 degrees. Beams that come together in a very flat angle may require a very long fin plate to provide a connection to the supporting girder and this can be quite awkward and require consideration of stability in the unsupported plate. The bending and shear capacity of the beam web with its flanges coped to make the connection will be considerably less than the basic beam capacity with the flanges in place and may pose problems during the connection design phase of the project. Figure 8.15 shows beam connections at three angles from the usual perpendicular connection to a 45-degree connection and finally a connection less than 45 degrees which produces very awkward connection plates.

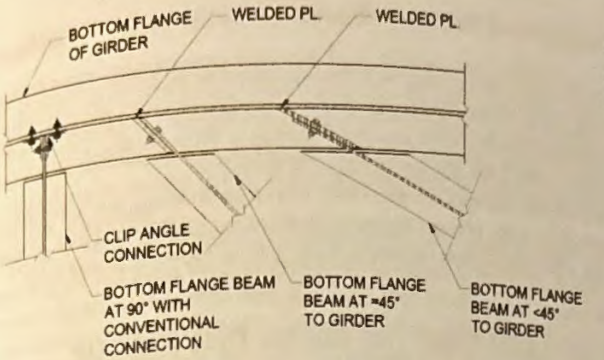


Figure 8.15. Effect of intersection angle on the connection

Solutions to this problem involve turning the deck direction or adding extra beams that connect to the girders at angles greater than 45 degrees, as shown in Figure 8.16.

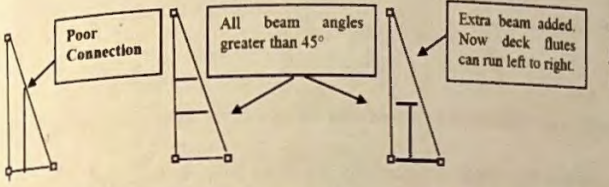


Figure 8.16. Options for avoiding low angle beam intersections

Attempt to Locate Structure Where It Best Suits the Architecture

Usually situating columns at the locations where the architect finds acceptable will drive the layout of much of the framing in a building. Keeping the framing simple in a complex architectural layout can be a challenge, but it is important for economy that there is as much repetition as possible. This is often a challenge that requires negotiation and patience. In schools, hospitals, hotels and other buildings where there are rooms off a central corridor it is usual to place columns in the corridor walls.

8.3 HOLES FOR MECHANICAL DUCTS AND OTHER SHAFTS THROUGH THE FLOOR

Holes for mechanical ducts and other vertical openings in the floor system should be framed with beams on all sides of the opening to provide support for the deck, as shown in Figure 8.17.



Figure 8.17. Several duct openings occurring in a tight area will require several beams to be added to make sure that the ends of the deck are supported at the edge of the opening. The beams will also assist with the support of the equipment that is above

Holes for larger mechanical ducts should be shown on drawings with approximate rough opening sizes provided but as mechanical engineers will often change the size or location of the duct without informing the structural engineer it is common and prudent not to dimension smaller openings. A note should be

added to the drawings: "For size and location of duct and other openings through the slab see mechanical and architectural drawings."

In terms of loads on the beams around the opening, no reduction should be made for portions of floor that is designed to be open. This gives an allowance for the weight of any mechanical equipment or shaft wall that may be supported in the duct area and gives structural capacity to fill in the hole and have it act as a floor should the duct be moved at the time of construction or in the future.

As it is very difficult to have all slab and roof openings fully shown on drawings, it is appropriate to include requirements in the general notes for trimmer beams to be added around floor and roof openings. The ideal situation is to have the girder larger than the beams it picks up, as shown in Figure 8.18.

8.4 AVOID GIRDERS SMALLER THAN THE BEAMS THEY PICK UP

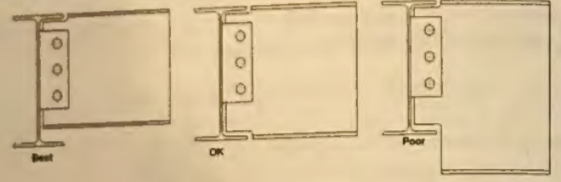


Figure 8.18. Options for size relationship between the beam and its supporting girder

This gives the easiest beam connection as only the top flange of the beam needs to be coped. Also acceptable is having the beam and the girder that it picks up of the same depth. This requires the beam to be coped at the top and bottom flanges but otherwise has no issues. A less desirable situation is the case where the beam is deeper than the girder that is picking it up. This may happen when the span of the girder is much less than the span of the beams and can satisfy force and deflection criteria without needing a section as deep as the beam. A girder that is less deep than the beam results in an awkward connection and will usually result in an RFI from the fabricator; it also looks strange in the field.

8.5 TRY TO USE W200 BEAMS OR LARGER TO SIMPLIFY CONNECTIONS

Sometimes a calculation on a short beam will show that only a W100 or W150 section is required, and while there are some locations where the use of beams of this size is very appropriate, the connection details will be more difficult for beams that are less than 200mm deep. Bolts in beams that are less than 200mm deep must be spaced horizontally rather than vertically. The connection is more robust and easier to execute if the bolts can be in a single vertical row, as shown in Figure 8.19. Moreover, the connection load eccentricity is reduced.

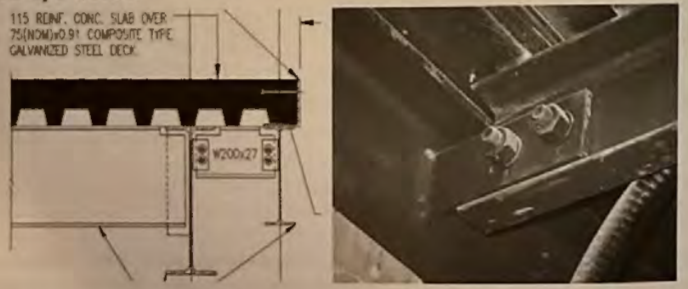
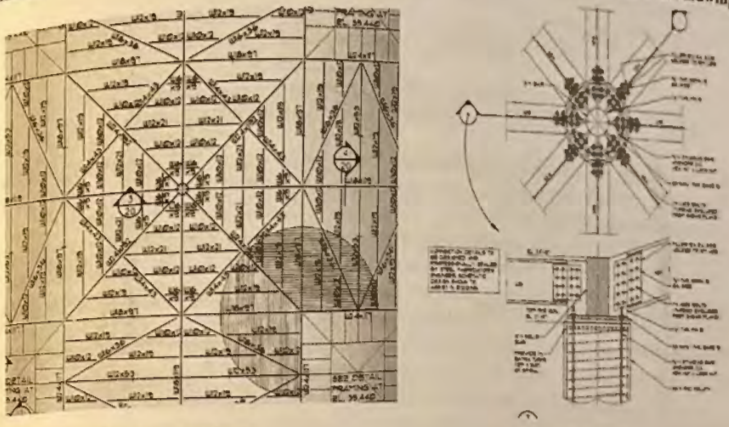


Figure 8.19. As shown in the photos above, the connection with the two bolts in a horizontal row has resulted in a long connection plate that should be checked for stability.

8.6 AVOID TOO MANY CONNECTIONS AT A SINGLE COLUMN

In typical rectangular framing there will be a maximum of four beams connected to a column and they are usually at right angles. This avoids conflict with flanges and makes for standard connections to the column. In framing circular areas or in areas where beams intersect the column at radial angles, there may be more than four beams intersecting a column or supporting girder. In these cases it is important for the structural designer to draw out the connection and determine if special detailing is required or even if the connection is possible.

Figure 8.20 shows the connection where eight beams come together at a single column; the plan view is shown at the left while the connection itself is shown at the right. As this was a truss arrangement and there were high axial forces present, this resulted in needing a solid section at the centre of the intersection so that the connection could be drawn out far enough that the flanges of the intersecting beams and column did not conflict. It is necessary to show this sort of non-standard detail on structural drawings.



Plan View of Roof Detail at Centre

Figure 8.20. Beam connection to a single column

8.7 AVOID TORSION ON OPEN SECTIONS

As open sections are weak in torsion, wherever possible designers should avoid framing that relies on the torsional capacity of I-beams, channels, planar trusses or other open sections for stability. If the torsional capacity of these members is required for the system to work, it will be flexible and may be subject to excessive deflection and vibration. If torsion is unavoidable, then closed sections such as HSS should preferably be used for the member subject to torsion and detailing of the end connection must include means by which the torsion in the section can be transferred to the support. The designer should be aware that the typical bolted web connection to a wide-flange beam would not be regarded as a suitable means of removing torsion from the beam.

Figure 8.21 shows the framing for the roof of a mechanical room where a curved edge is required. The beam at the edge cannot act as the girder because its curved shape will require it to have a torsional ability that it does not possess. Framing using a straight girder below the roof beams and using a curved beam only as a facia support is the appropriate structural solution in this case.

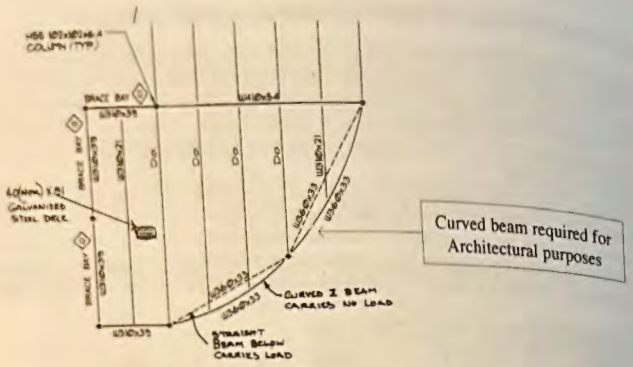


Figure 8.21. Beam connection to a curved edge

A well laid-out design will provide framing that accommodates unbalanced loading without having to rely on torsion of open sections. For example, a canopy supported on a single support at the centreline may be desirable architecturally but does not work well unless there is equal loading on both sides of the canopy or unless a stiff HSS is used as the support beam. Two options are shown in Figure 8.22.

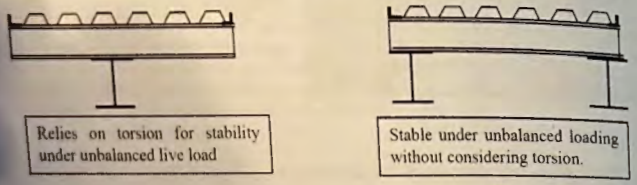


Figure 8.22. Beam connection to a curved edge

In the case of partial-height façades supported by a spandrel beam, the floor slab and a diagonal strut ("kicker") can combine to develop a force couple that resists the torsion that would otherwise be applied to the beam. Alternatively, a perpendicular "roll beam" that resists the spandrel torsion in flexure (note that the connection between the spandrel and roll beams must resist moment) can be used. Information about designing façade supports can be found in AISC's *Design Guide 22*.

8.8 ADJUSTING THE FRAMING TO ACCOMMODATE SNOW DRIFTING

The snow drifting loads will often cause the deck to be overstressed unless the span of the deck is reduced by adding extra beams. It usually would not be economical to have the beam spacing the same as required in the snow drift area throughout the entire roof so the designer is left with the option of adding extra beams for the snow drift area or changing the spacing of the beams in the bays affected by the snow drifting. Two options are shown in Figure 8.23. In Chapter 7 when we looked at roof decks, we examined the computation of deck capacity in drift areas. Note that the drift can extend from the higher roof in any direction including on the diagonal.

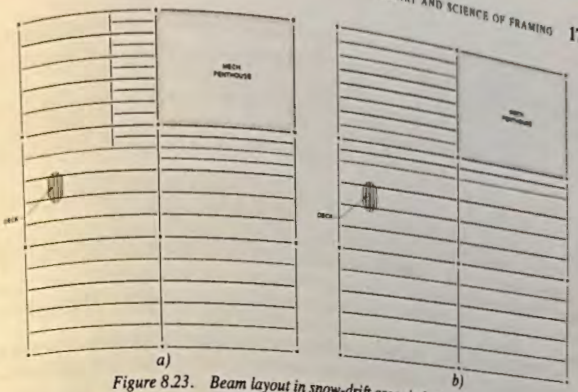


Figure 8.23. Beam layout in snow-drift area; in both cases the beams are spaced closer together in the areas of snow drift

8.9 SUPPORTING STEEL COLUMNS ON BEAMS

Frequently it is necessary to support a column on a beam to create a column transfer. When this occurs, special details are necessary to prevent local and perhaps global problems in the supporting beam.

S16 covers the local resistance of beams due to point loads from above in Clause 14.4, requiring web stiffeners when the load from the column exceeds the capacity of the web in crippling and yielding. While it may be acceptable to use no stiffener under the column from a code standpoint, it is good practice to place a web stiffener under all columns that land on beams. In addition to assisting the web, the addition of a vertical stiffener prevents undesirable behaviour including stability issues.

The bearing resistance of an unstiffened web is given by Clause 14.3.2, which covers web crippling and yielding. The bearing resistance depends on the location of the load (end of beam or interior load), the thickness of the flange and the thickness of the web.

For an **interior load**, the capacity is the lesser of:

i) $B_r = \phi_{bi} w(N + 10t) F_y$

ii) $B_r = 1.45 \phi_{bi} w^2 \sqrt{F_y E}$

where $\phi_{bi} = 0.80$, N = length of bearing, w is the web thickness and t is the flange thickness of the supporting beam.

For a W410x39 in 345MPa yield steel with a 150mm bearing length (N = 150mm), (W410x39: w = 6.4mm t = 8.8mm)

$$B_r = \phi_{bi} w(N + 10t) F_y = 0.8 * 0.0064m * (0.150m + 10 * 0.0088m) * 345{,}000 \frac{kN}{m^2} = 420kN$$

$$B_r = 1.45 \phi_{bi} w^2 \sqrt{F_y E} = 1.45 * 0.8 * (0.0064m)^2 * \sqrt{345000 \frac{kN}{m^2} * 200 * 10^6 \frac{kN}{m^2}} = 395kN$$

The governing capacity is 395kN. This magnitude of load is certainly enough for the reaction under most joists (stiffeners are seldom placed under point loads from joists) and would cover many one-storey columns. However, regardless of the numbers some design offices will consider it mandatory to add stiffeners under column loads as good practice. It is worth keeping in mind that the two most significant structural

failures in Vancouver were the collapse of the Second Narrows bridge during its construction in 1958 and the collapse of the Metrotown Save-On-Foods store on opening day in 1988, and both these failures were largely due to failures of an unstiffened wide-flange beam web at a load concentration.

For stability it may be desirable to have a beam added perpendicular to the main beam at the point where the column is picked up. This assists in preventing the column from twisting the transfer beam. This is of particular importance during structural steel erection when the top of the column is not stabilized and the deck is not stabilizing the transfer beam.

Especially if the column is picked up near the end of the beam the drawings should clearly show the factored shear that the steel detailer is to use when designing the beam connection.

The support for the column will usually be a base plate that is bolted to the beam. To assist in erection, S16 requires there be 4 bolts between the beam and the column base plate. As holes for the two bolts will remove approximately 44mm from the top flange of the beam, it may be necessary to increase the size of the beam to account for the bolt holes if the flange is less than 300mm wide (15% of 300mm is 45mm and removal of 15% of the flange for holes is usually ignored; see Clause 14.1).

For lightly loaded columns, a three-quarter height web stiffener is all that is necessary unless the beam is sandwiched between a column from above and below, in which case a full height stiffener is always used.

Clause 14.4 of S16 has some detailing requirements for bearing stiffeners requiring that they be detailed as columns. Width-to-thickness ratios of bearing stiffeners should conform to Clause 11. The requirements to satisfy Class 2 using F_y = 300MPa plate for the minimum stiffener would be (b/t)<340/ Sqrt (F_y), resulting in (b/t <19.6), which gives stiffener size as shown in Table 8.12.

Table 8.12. Flange-width and stiffener size

Flange-Width	Overhang	Stiffener
235mm	117mm	6mm
314mm	157mm	8mm

Stiffeners may need to be larger than this depending on load conditions from the column being supported. Figure 8.24 shows stiffeners added to a transfer beam at the column location to assist with the load transfer along with the four bolts to connect the column base plate, which may require choosing a sufficiently wide flange on the supporting beam.

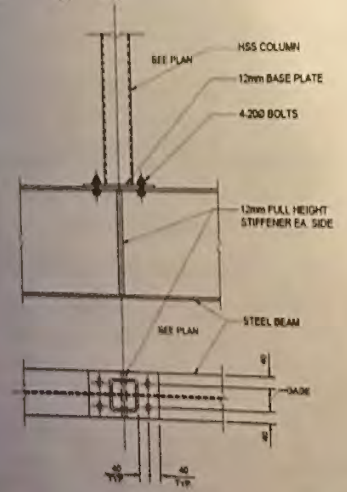


Figure 8.24. Column supported on beam with full-height stiffener

8.10 AVOID COLUMN TRANSFERS

Column transfers are always expensive, both from the standpoint of the fabricator and the designer, and they are best avoided if possible. Conversely, architects are usually unreceptive to suggestions of placing columns inside their rooms and a change of use in a building from one level to another may leave no option but to include a column transfer. Transfers will usually involve deeper and heavier beams and require detailing such as shown in Figure 8.24 for picking up the column. Configuring the framing to avoiding the column transfer would be advantageous. For example, in a school it may be that running the columns down the corridor walls is better than running them in the walls between rooms, especially if the room size varies; this would require running the beams or joists perpendicular to the corridor but would eliminate the need for a column transfer where the room size changes.

8.11 STEPS IN FLOORS OR ROOFS

Many roofs will have small changes in elevations that are considerably less than a full storey height. Roof steps usually involve placing a beam or girder under both the upper and lower roof edges, as shown in Figure 8.25a. Sometimes the roof step is of a sufficiently small size that the upper and lower edge beams would conflict. A solution where a single beam picks up both the roof levels may be preferable and is shown in Figure 8.25b. In some cases, even if the two beams do not conflict it may be preferable to deepen the edge girder at the top of the roof and have a single beam giving the same solution as shown at the right.

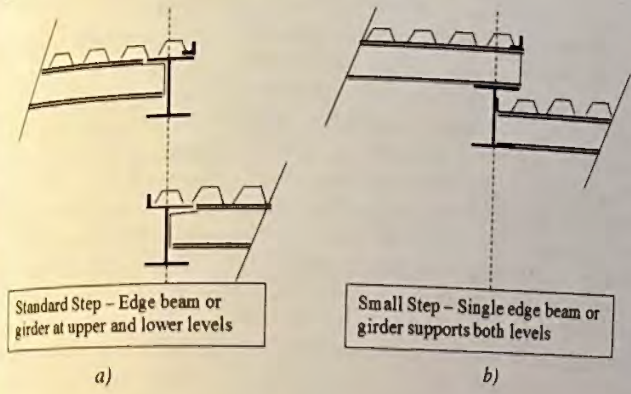


Figure 8.25. Beam placement in roof steps

8.12 AVOID FRAMING THAT "GOES IN A CIRCLE"

Framing for steel floors should be designed such that beams can be lifted into place one at a time and connections provided to only two ends at one time. Sometimes a framing problem will arise where a designer has produced a framing system that cannot be erected in this way and requires that several elements be erected at the same time. Such a system is shown in Figure 8.26a, where an opening in the floor has resulted in the designer producing the awkward framing system shown. In this system the loads "go in a circle," as the load on Beam A transfers partly to Beam B, which in turn transfers the load to Beam C, which then transfers part of the load to Beam D, which in turn passes the load back to Beam A, as shown in Figure 8.26b. It is important to note that beyond the fact that erection of the system as designed would be complex, it also is unsound from a structural standpoint.

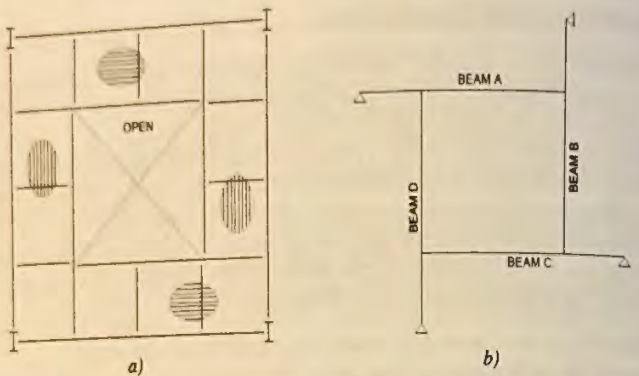


Figure 8.26 Beam framing that "goes in a circle"

Framing of this sort will need to be changed during the shop drawing stage, possibly resulting in additional charges.

8.13 BEAMS LOCATED IN WALLS

Architecturally it may be desirable to install beams in walls, but this has two effects:

1) The flange width of a beam to be located in a wall is limited to 25mm less than the stud width intended to be used in the wall so that tbe final wall with its steel stud and gypsum board construction will be straight, even if there is a slight variation in the placement of the structural columns. For example, a W310x31 beam with a flange width of 164mm might be the most economical section to use, but if it is to be located in a 6-inch (152mm) stud wall a W360x33 beam with a 127mm flange would be selected. As there are very few deep beams with narrow flanges there is a limit to the depth of beams that will fit in walls; for example, the smallest flange on a W410 beam is 140mm. Flanges of beams are often buried in walls for the perimeter of low-rise buildings, and these walls usually have several columns on the outside gridline. While the structural columns will have some variation from the outside gridline it is still desirable to have a straight outside wall and this is achieved by keeping the steel studs forming the wall exactly centred on the outside gridline. Using a beam with a 140mm flange width in a stud wall with 152mm studs leaves very little ability for the steel stud contractor to adjust the position of the studs if the structural beam that supports the floor or roof above is not in exactly the right location.
2) If the upper track of the studs is attached to the bottom flange of the beam that is to be buried in an exterior wall, the studs can transfer wind forces to the bottom flange of the beam and produce out-of-plane bending. An interaction calculation is required between the moment caused by vertical loads on the beam and the lateral forces from the wind on the studs. Out-of-plane deflection of the bottom flange should also be computed. It may be necessary to stiffen the bottom flange of the beam by adding angles to the bottom flange or it may be necessary to decrease the column spacing at the edge of the building. In cases where the bottom flange is trying to pick up wind loads, the bottom flange of the beam should be attached to the column with clips (call up on drawings) such that the wind force in the bottom flange can be extracted by the column, as shown in Figure 8.27.

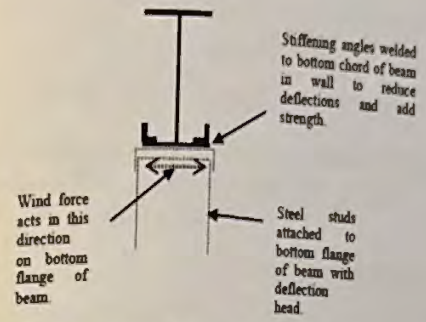


Figure 8.27. Bottom flange of beam picks up wind loads

Note that when trying to fit beams in stud walls (a practice common in many low-rise steel buildings including schools) the spacing of the exterior columns may need to be reduced to give a beam flange that is sufficiently narrow for the wall stud size selected by the architect. Instead of adding the angles to the beam as shown in Figure 8.27, it may be more economical to use a heavier beam as shown in the following example.

EXAMPLE—DESIGN AN EXTERIOR WALL BEAM

A W310x31 beam is located in an exterior wall where the wind load on the studs, including interior and exterior pressure, is 1.25kPa. The tributary height of the studs is 3.5m / 2 = 1.75m and the wind load is transferred to the bottom flange of the beam. Compute the combined stress caused by the factored wind and vertical loads and compute the deflection under service wind load. The columns are spaced at 4.0m. The vertical dead load on the beam is 4.5kN/m and the live load is 5.3kN/m. A loading diagram is shown in Figure 8.28.

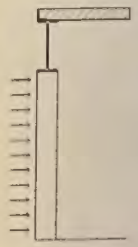


Figure 8.28. Loading diagram of wall system

The factored wind load = 1.75m * 1.25kPa * 1.4 = 3.06kN/m

Note that the load factor for wind load is 1.4.

Factored moment = 3.06kN/m * $(4m)^2$ / 8 = 6.1kNm

The S for the bottom half of the beam is S_y / 2 = 36.7x$10^3$$mm^3$ / 2 = 18.4 x$10^3$$mm^3$

(W310x31 is a Class 3 beam so using Z would not be appropriate.)

$M_r = \phi * F_y * S_y = 0.9 * 345{,}000kN/m^2 * 18.4 \times 10^{-6}m^3 = 5.71kNm$

This is obviously unacceptable considering the resistance in the out-of-plane condition is less than the applied-load moment and we would need to consider the simultaneous effect of vertical loading which will result in biaxial bending.

One option would be to add angles to the flanges of the beam, but as a first step consider using a beam with more capacity to resist the wind forces. Try W310x39 with a 165mm flange width.

The Z for the bottom half of the beam is $Z_y / 2 = 135 \times 10^3 mm^3 / 2 = 67.5 \times 10^3 mm^3$

(W310x39 is a Class 1 section so Z would be appropriate).

$M_{rx} = \emptyset * F_y * Z_y = 0.9 * 345{,}000 kN/m^2 * 67.5 \times 10^{-6} m^3 = 20.9 kNm$

The beam is under biaxial bending from the wind acting at the same time as the vertical loads, so this would need to be combined with the moment in the stiff axis from vertical loading using the requirements of Clause 13.6(f).

Moment from vertical loading during maximum wind

$= 1.25D + 0.5L = 1.25 * 4.5 * (4^2/8) + 0.5 * 5.3 * (4^2/8) = 11.25 kNm + 5.3 kNm = 16.6 kNm$

$$\frac{M_f}{M_r} + \frac{M_f}{M_r} < 1 \quad \frac{6.1 kNm}{20.9 kNm} + \frac{16.6 kNm}{189 kNm} = 0.29 + 0.09 = 0.38 \text{ (OK<1)}$$

Deflection calculated using the W310x39 and the 0.75 SLS factor for wind loading.

$$= \frac{5 * W_l * L^4}{384 * E * I} = \frac{5 * (1.25 kPa * 1.75 m * 0.75) * 4.0^4 * 1000}{384 * 200 * (7.27 / 2)} = 7.5 mm = L / 531$$

Conclusion: Stress and deflection are OK if W310x39 is substituted for the W310x31.

8.14 TRY TO KEEP FRAMING SIMPLE AND USE REPETITIVE ELEMENTS

To achieve economy in steel framing it is desirable to have repetition in members, and a reasonably conventional and preferably simple layout of members. One example of a structure where this was not able to be achieved is shown in Figure 8.29. The roof was desired to be dramatic and a series of intersecting cones was architecturally conceived that would have achieved this aim in a very elegant manner. At each side of the roof a series of barrel vaults would have been visible and there would have been a dramatic contouring of the top surface of the roof.



Figure 8.29. Roof as architecturally conceived – set of intersecting cones

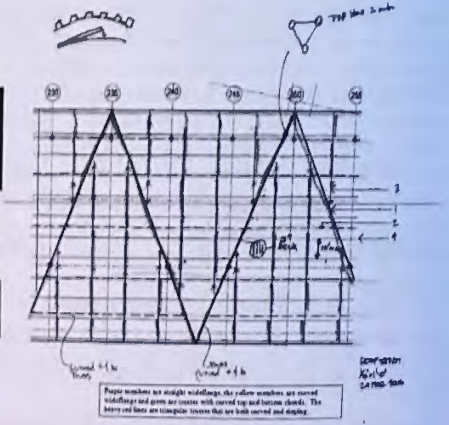


Figure 8.30. Resulting schematic design of roof system

Framing conceived for the roof of Figure 8.30 had smaller members as a mix of straight and curved wide-flanges, and custom trusses with curved top and bottom chords. The smaller members were to be supported on triangular trusses that were both curved and sloping.

Not surprisingly, the lack of repetition and complexity of the roof made the solution too expensive for the project budget. To keep some of the interest of the roof and not go completely back to a flat roof we proposed a series of barrel vaults, which was considerably less complex and had more repetition and was deemed to be possible within the project budget. This is shown in the Revit drawing in Figure 8.31. The resulting barrel-vault roof is shown in Figure 8.32.

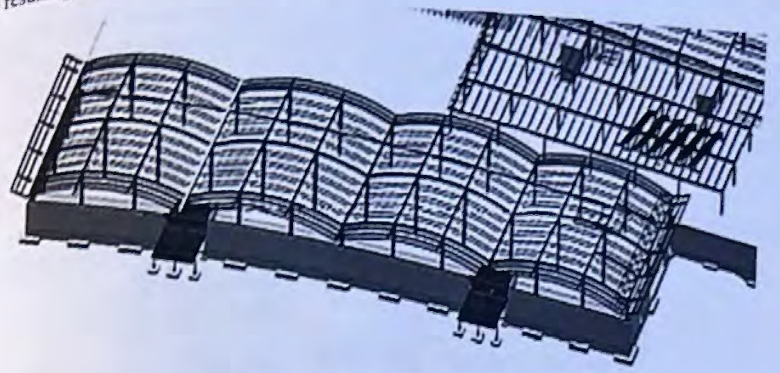


Figure 8.31. REVIT model of barrel vault roofs



a) *b)*

Figure 8.32. Barrel vault roofs following steel erection but before the architectural finishes are added. While the framing did not fully satisfy the architect's desire for a roof with intersecting cones it did produce a solution that was more dramatic than a flat roof and fit within the project budget. When doing curved roofs, the deck and support beams should be laid out such that the deck is bent the "easy way" and the structural members are bent to give the curved profile

8.15 APPROPRIATE INPUT LOADING FOR VIBRATION ANALYSIS

Vibration is one of the limit states for structural steel and should be an important consideration in the design of the structure. A floor that vibrates too much can be very annoying and cause considerable discomfort to the users. The analysis of a floor for vibration involves three stages, all of which have considerable variation in the approach and parameters used.

1) The input loading
2) The model including the damping
3) The acceptance criteria.

The input loading for the vibration can be continuous due to rhythmic activity such as aerobics in a fitness club. Machinery or passing traffic or trains can also cause continuous vibration. The source of vibration can also be transitory, in which the vibration decays away; this type of vibration can be typified by floor excitement due to footsteps and walking. The live load that is associated with this dynamic loading is always considerably less than the design live load for the floor.

8.16 SELECTING THE DAMPING TO BE USED WITH VIBRATION ANALYSIS

Damping can have a huge effect on the perception of vibrations and the resulting annoyance to building occupants. In a heavily damped system the effects of walking vibration will quickly disappear, while in a lightly damped system the vibration may continue for some time. This is demonstrated in Figure 8.33, which is taken from output of FEMA's educational program *Nonlin*. Unfortunately, many steel floors—especially those without softening finishes or deadening partitions—will be very lightly damped, with vibrations such as those set up during walking taking a long time to dampen out. Damping will have a significant effect on structures that are close to resonance with the forcing function of a continuous excitation such as rhythmic activity.

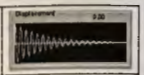
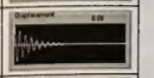
Bare Floor	3% Damping	
Finished Floor, ceiling ducts, flooring and furniture	6% Damping	
Finished Floor with partitions	12% Damping	

Figure 8.33. Compared damping of vibration for various floor types
Source: FEMA's educational program *Nonlin*

8.17 FLOOR VIBRATION ANALYSIS METHODS

Basic analysis of the floor frequency and acceleration of the floor can be found by following the procedures outlined in *Floor Vibrations Due to Human Activity*, AISC-CISC Steel Design Guide Series 11.

EXAMPLE—FLOOR VIBRATION ANALYSIS

Floor vibration analysis often consists of a spreadsheet analysis using the beam and girder properties. These spreadsheets can be developed using the examples found in the listed references. The example shown in section 8.18 is from a spreadsheet developed to address the vibration calculations quickly. The aim is to find the effective frequency and acceleration of the floor such that it can be compared to acceptance criteria. The results of that analysis are as shown in Table 8.13, as compared to the W460x60 beam that would be selected if composite behaviour was not assumed.

Table 8.13 Floor vibration analysis

	W460x60	W410 x 39 Composite Design
Floor Frequency	5.25 Hz	4.80 Hz
Acceleration	0.533% g	0.667% g

Source: AISC-CISC vibration manual

Note that the compositely designed beam has a lower frequency (thus longer period) than the non-compositely designed beam. The compositely designed beam has a higher acceleration than does the non-compositely designed beam.

A stiffer floor vibrates naturally with more cycles per second. A more flexible floor will have a greater amplitude of vibration and will often be more perceptible to the users.

8.18 CRITERIA FOR DETERMINATION OF ACCEPTABILITY OF VIBRATION

The perception of floor accelerations varies with the frequency of the floor and the vibration. To determine if a floor will have an undesirable vibration environment, comparison is made to charts that plot acceleration and frequency. The perception of and annoyance due to vibrations will vary depending on the type of use. Floor vibrations on a footbridge or a steel stair are less objectionable than in an office or apartment building where occupants are at rest. The perception of vibrations will also vary depending on the damping: vibrations that quickly dampen out will be less objectionable than those that keep on going for a long time or have the opportunity to amplify if the source of vibration continues. To take account of this, on the acceptability chart in Figure 8.34, plot both the acceleration and floor frequency for various activities. The compositely designed W410x39 beam is plotted to show where it fits.

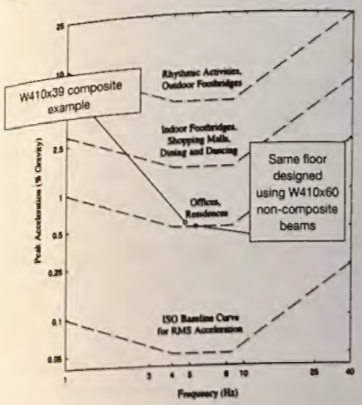


Figure 8.34. Recommended peak acceleration for human comfort for vibrations due to human activities
Source: Allen and Murray, 1993; ISO 2631-2: 1989

8.19 LATERAL VIBRATIONS OF WALKWAY STRUCTURES

The material on floor vibrations usually focuses on vertical vibrations; however, there have been some well-publicized examples of structures that have "failed" as the lateral movements due to walking have made the structure too uncomfortable to use. One such structure is London's Millennium Bridge, which was opened and then immediately closed until additional damping could be added to stop the lateral swaying. The websites listed in the reference at the end of this book provide a good background to the Millennium Bridge vibration issues. The design engineers have demonstrated an excellent example of crisis management by creating a website that explains how their engineering analysis solved the problem. While vibrations are generally considered to be an annoyance, they can also cause discomfort and alarm and even death. In November 2010, over 350 people were killed in Phnom Penh, Cambodia when a stampede occurred on a crowded bridge that had begun to laterally oscillate from the numbers of people walking on it.

Sample Floor Frequency Analysis

CONCRETE SLAB ON STEEL BEAM FLOOR FREQUENCY		
File:	SteelFloorFreq2.xls	
Date	April 6, 2008	
Time:	14:32:03	
	Follow CISC Steel Design Guide Series Floor Vibrations due to Human Activity (1997)	
Loading Information		
LL=	**0.5**	Live load at time of analysis (KPa)
Beta=	**0.03**	Damping ratio 0.02 for shopping malls 0.03 for office
Po=	**0.29**	Constant force =.29 kN for offices and shopping malls
SIDL=	**0.2**	Superimposed Dead Load (KPa) (not incl. beam / slab)
Steel Beam Information		
L=	**10.5**	Beam length (m)
S=	**3**	Beam Spacing (m)
As=	**6640**	Beam Area (Sq mm)
Ix=	**212**	Beam Inertia (mm^4 / 1e06)
Mb=	**52**	Beam Mass (Kg/m)
db=	**450**	Beam Depth (mm)
Sg=	**9**	Girder Span (m)
Wg=	**74**	Girder Mass Kg/m
Hg=	**529**	Height of girder (mm)
Ig=	**410**	Inertia of girder (steel only)
Ag=	**9490**	Girder Area of steel (sq mm)
Es=	**200000**	Modulus for Steel (MPa / 1,000)
Concrete Deck Information		
dt=	**80**	Topping Depth (mm) to flute top
Dd=	**50**	Deck Depth (mm)
Dconc=	**18.15**	Concrete Density (KN / cu m)
Fc=	**30**	Concrete Strength (MPa)
Ec=	**16000**	MPa (use 0 if to be computed from F'c)
Sweight-=	**2.0**	Slab Weight + Deck Weight (KN / sq m)
Computation of beam section properties		
Dslab=	**130**	Total Depth of slab (mm) (topping + ribs)
Ec=	**16000**	Modulus for Concrete (MPa / 1,000)
Sw=	**204**	Surface density of slab (Kg /sq m)
n=	**9.26**	Dynamic Modular Ratio of concrete

Computation following vibrations due to walking (Follows method of Allen & Murray 1993) See also example 4.3 in CISC guide to floor vibrations (1997)		
Beam Mode:		
Slab Wid=	3	Effective slab width (m)

Computation of beam section properties (Using only concrete above flutes)		
Dslab=	130	Total Depth of slab (mm)
Ec=	16000	Modulus for Concrete (MPa / 1,000)
Sw=	204	Surface density of slab (Kg /sq m)
n=	9.26	Dynamic Modular Ratio = Es/ (1.35*Ec)
Deff=	80.0	Depth of slab (mm) above flutes
Dst=	8.6	Transformed slab depth (mm)
Ac=	25920	Transformed concrete area (sq mm)
Yc=	540.0	Distance from bottom of beam to C.L. Concrete
Yb=	475.8	Centre of gravity transformed beam from bot flange
It1=	14	Itransformed part 1 (inertia of transformed slab)
It2=	107	Itransformed part 2 (Offset of concrete area)
It3=	212	Itransformed part 3 (beam inertia)
It4=	418	Itransformed part 4 (Offset of steel beam area)
It=	750	Transformed inertia (total of parts 1 to 4)
w=	1.5	Supported weight (Live load) (KN / m)
w=	0.6	Supported weight (Superimposed Dead load) (KN /m)
w=	0.5	Supported Weight (Beam) (KN / m)
w=	6.0	Supported Weight (Slab) (KN / m)
w=	8.6	Total Weight supported (KN / m)
Delta Bm=	9.08	Deflection of beam in mm
Fj=	5.92	Beam frequency in Hz
Avg CT=	105	Average concrete thickness (mm)
Ds=	10	Transformed moment of inertia slab direction. m^4/m x 1E06
Dj=	250	Transformed moment of inertia beam direction. m^4/m x 1E06
Bj=	9.49	Effective beam panel width (m)
Wj=	429	Weight of beam panel (KN)

Computation of girder section properties (Using only concrete above flutes)		
Slab Wid=	3.6	Effective slab width (m)
Dslab=	130	Total Depth of slab (mm)
Ec=	16000	Modulus for Concrete (MPa / 1,000)
Sw=	204	Surface density of slab (Kg /sq m)
n=	9.26	Modular Ratio = Es/ (1.35*Ec)
Deff=	80.0	Depth of slab (mm) above flutes
Dst=	8.6	Transformed slab depth (mm)
Ac=	31104	Transformed concrete area (sq mm)
Yc=	619.0	Distance from bottom of beam to C.L. Concrete
Yb=	536.1	Centre of gravity transformed beam from bot flange
It1=	17	Itransformed part 1 (inertia of transformed slab)
It2=	214	Itransformed part 2 (Offset of concrete area)
It3=	410	Itransformed part 3 (beam Inertia)
It4=	700	Itransformed part 4 (Offset of steel beam area)
It=	1340	Transformed Intertie (total of parts 1 to 4)
wg=	30.9	KN / m Girder Loading
Delta Bm=	9.83	Deflection of beam in mm
Fj=	5.68	Beam frequency in Hz
Cg=	1.8	Beams supported to girder web.
Dj=	250106	mm^3
Dg=	127656	Girder trans mom of inertia Ig/ Lg for interior girders
Bg=	19.2	
Bg kick=	21.1	max of ⅓ times floor length
Bg use=	19.2	Lesser of two above
Wg=	507	Girder panel weight

Combined Mode Properties		
Dg'=	9.33	mm Reduce girder deflection if girder span less than joist panel width
Dj=	9.08	mm
fn=	4.15	Hz Floor fundamental frequency
W=	212	KN Beam mode weight
W=	257	KN Girder mode weight
W=	468	KN Combined mode panel weight
Beta W=	14.1	KN
ap/g=	0.482%	Percent g

CHAPTER 9
COLUMNS, BEAM-COLUMNS, AND FRAMES

This chapter covers members that are loaded with an axial force and in some cases with an axial force and a moment. In most buildings the members with axial loads are the columns but axial loads can also be present in truss members and in some frame members and elements such as braces carrying wind and seismic loading. The failure method of columns will often be a sudden buckling so there is a need to design these elements correctly to prevent this failure. We will also cover some of the Building Code requirements necessary to determine forces on the columns we design.

9.1 COLUMN DESIGN

1) The following are guidelines for the design of steel columns for buildings in Canada. These guidelines refer to columns with low moment that are not part of the lateral resisting system. As moment frames are used infrequently, this covers the vast majority of columns in low-rise buildings. The basic process is:
2) Determine axial load in column using appropriate requirements from the Building Code. The forces on the column should include load factors and live load reductions.
3) Determine what, if any, moments are present; if there are significant moments present, then the design should be performed as a beam-column. If the beam connections are designed to carry reaction shears to the column centrelines, then the column moments are often taken as being zero.
4) Determine the unsupported length of the column using Clause 10.3 of S16.
5) Using the column capacity tables (green pages) of the *CISC Handbook*, select a column section that carries the required axial load for the unsupported length.

Design the base plates.

- The following points should be kept in mind:
- Use only Class C HSS sections. Do not use Class H sections (or the tables for Class H sections) as these sections have a much longer delivery time and greater cost than Class C sections.
- Where possible, use square or circular HSS sections in preference to rectangular HSS sections as orientation of placement is not an issue with a symmetric section and the column cannot be installed in an incorrectly rotated orientation.
- If using rectangular HSS columns or wide-flange columns, then the KL/r ratio about both axes must be evaluated. This is especially important if there is support in the weak axis at mid-height, making it uncertain if weak axis or strong axis buckling will govern.
- Call up a cap plate at the top of HSS columns to prevent water getting in. Water may lead to internal rusting or tube spitting under freezing conditions.
- KL/r must be less than 200. When using column capacity tables (green pages) of *CISC Handbook*, this is automatically taken care of as axial values for lengths that would produce a KL/r of greater than 200 are not listed.
- When using HSS columns be aware of the different properties for ASTM A500 and G40.20 steel as the A500 sections are weaker than the difference in yield strengths (345MPa or 317MPa instead of 350MPa) would indicate. This is because the tolerance standards in producing the sections for

A500 differ from those used when producing G40.21 sections and wall thickness for an A500 section may be less than that of a similar G40.21 section, producing strength losses up to 10% for similar sections. See example in Table 9.1.

Table 9.1. Differences between HSS sizes and materials

Size	Material	F_y	Area (sq mm)	r (mm)	C_r for L_u=4m
HSS 102x102x6.4	G40.21	350MPa	2320	38.4	293kN
HSS 102x102x6.4	ASTM A500	345MPa	2110	38.7	268 kN
HSS 102 Dia. X 6.4	G40.21	350MPa	1900	33.8	199kN
HSS 102 Dia. X6.4	ASTM A500	317MPa	1720	34.0	177kN

- Your design should reflect the availability of materials. Round HSS sections are very difficult to obtain in G40.21 and should only be designed in ASTM A500. Square or rectangular sections can be G40.21 or ASTM A500.
- Columns supporting fire-rated assemblies may be governed by M/D ratios (mass over perimeter ratios) as that is often a criterion in fire rating. For columns supporting fire-rated assemblies, review the architectural rating requirements and increase column weight as required to give the required fire resistance. (See discussion in Chapter 1.)
- Columns in single-storey buildings that are primarily loaded with axial load (i.e., they are not part of a moment frame) will be lighter when designed with HSS members than when designed with wide-flange sections. However, HSS material is more expensive on a weight basis than wide-flange sections.
- HSS sections are often used for exposed columns as they have a cleaner shape than the wide-flange section, have less painting to cover them and will be easier to keep clean.

EXAMPLE—DESIGN OF A STEEL HSS COLUMN

Design an interior HSS column for a single-storey building in Vancouver for the load combination 1.25D + 1.5S + 0.4W using the loads given in Table 9.2. Columns are spaced at 9m in one direction and 10m in the other.

Table 9.2. Roof loading Project X in Vancouver (120m x 100m flat roof)

Load Source	Dead (kPa)	Snow (kPa)	Down Wind (kPa)	Factored (kPa)
Vancouver Roof snow (Ss=1.8 Sr=0.2, Cb=0.93)		1.87		2.81
Interior suction and positive exterior			0.39	0.16
Roofing / Insulation / Ceiling allowance	0.90			1.13
Steel Deck	0.10			0.13
Total load for Deck Design	**1.00**	**1.9**	**0.4**	**4.22**
Suspended Mechanical / electrical	0.20			0.25
Joists at 2000 o/c	0.10			0.13
Total load for Joist Design	**1.30**	**1.9**	**0.4**	**4.59**
W530x72 Girder at 9000 o/c	0.08			0.10
Total load for Girder and Column Design	**1.38**	**1.9**	**0.4**	**4.69**

Tributary area = 9.0m x 10.0m = 90m²

The factored load is 4.7kPa computed using 1.25D + 1.5S + 0.4W (1.25 * 1.38 + 1.5 * 1.87 + 0.4 * 0.4 = 4.7kPa). Note that there is no area reduction for the wind or snow loading.

Factored Column load = Factored area loading * Tributary area

= 4.7kN/m² * 90m² = 423kN

Figure 9.1 shows column dimensions that need to be considered for length calculation.

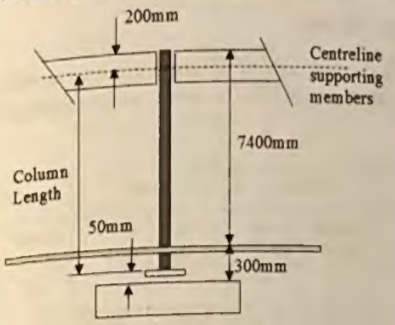


Figure 9.1 Column showing dimensions that need to be considered for length calculation

Calculate the column length: Column length is given in S16 Clause 10.3.1 and requires that the unsupported length of a column is from the centre of the restraining members to the top of the base plate.

Height from U/S of Deck to top of slab on grade	=	+7.40m
Add top of Slab on Grade to top of footing	=	+0.30m
Subtract 25mm grout and 20mm base plate	=	–0.05m
Subtract U/S deck to centre supporting members	=	–0.20m
		7.45m

This type of column is considered pin-ended: KL_u = 1.0 * 7.45m = 7.45m. Say, 7.5m

Using the column capacity tables (green pages) of the *CISC Handbook* with KL_u = 7.5m and C_f = 423kN we find the following columns satisfy the criteria:

HSS 152x152x9.5	C_r = 458kN	Weight = 40.9kg/m
HSS 178x178x6.4	C_r = 502kN	Weight = 33.4kg/m
HSS 203x203x6.4	C_r = 700kN	Weight = 38.1kg/m

The *CISC Handbook* column capacity tables list L_u of 7.2m and 7.6m. Consequently, you can interpolate, calculate exactly using the formulas or be quick and just use the values for 7.6m as was done in the preceding calculation. Unless there is a pressing reason to use the 152x152 section the 178x178 section would be selected as having the least weight. Note also this section has excess capacity over the 152x152 section.

EXAMPLE—DESIGN A STEEL WIDE-FLANGE COLUMN

Design a steel wide-flange column for the problem of the previous example.

Axial load is unchanged (P_f = 423kN) and KL_u is unchanged (7.5m)

Using the column capacity tables (green pages) of the *CISC Handbook* we find the following possible sections:

W200x46	C_r = 431kN	Weight = 46kg/m
W250x49	C_r = 430kN	Weight = 49kg/m

The most economical section (in terms of weight) that satisfies the load criteria is the W200x46. The most economical wide-flange column is 40% heavier than the most economical HSS column.

Wind uplift on the roof should also be considered with a viable load path provided. The load case of interest is factored net uplift, which is 1.4 (wind uplift) – 0.9 (dead). If the roof supported by the column with its 9m x10m tributary area, had a factored net wind uplift of 1.0kPa, then the factored net uplift to be resisted by the column would be 1kPa * 10m * 9m = 90kN. This would require each anchor rod resist 90kN/4 = 22.5kN and the footing weight would need to be 90kN/(23.5kN/m³ * load factor = 0.9) = 4.25m³ of concrete. This may well govern the size of the footing.

Columns Picking Up More Than One Storey

Columns in buildings with suspended floors must carry the load of the suspended floor as well requiring us to perform a column takedown. The column takedown will often be similar to that shown in Table 9.3.

Table 9.3. Column takedown

COLUMN A-12			DEAD		LIVE		SNOW	
		Tributary Area (m²)	q dead (kPa)	Dead (kN)	q live (kPa)	Live (kN)	q snow (kPa)	Snow (kN)
Roof	9m x 7m	63	1.2	76			1.58	100
Level 2	As above	63	3.8	239	2.4	151		
Column				4				
LL Reduction (For Level 2 Only)						-46.2		
	Sum=			319		105		100

Using the NBCC load combinations studied in Chapter 2 when we discussed load combinations, we get the following factored load at the bottom of the column:

$$C_f = 1.25D + 1.5L + 1.0S = 1.25*319kN + 1.5*105kN + 1.0*100kN = 656kN$$

or

$$C_f = 1.25D + 1.0L + 1.5S = 1.25 * 319kN + 1.0 * 105kN + 1.5 * 100kN = 654kN$$

The Building Code load combination Table 4.1.3.2.A shows we don't have to consider both snow and downward wind as simultaneous companion loads. Downward wind will therefore usually be ignored from column takedowns unless it is greater than the snow load.

For columns in buildings that are only two or three storeys high, it is common for the steel fabricator to construct and erect the column as a single element and thus avoid column splices at each floor. In low buildings, it is common to have columns of constant cross-section throughout the height, even if the section required at the base is larger than the section required at the underside of the roof. The column size required at each level is a function of the unsupported length and the axial load. If unsupported lengths are the same for all storeys, then it is possible to design the column using only the axial load at the column base. However, if the unsupported length is greater at the lift to the roof than at the lower floors, then a check on the axial capacity should also be made there.

K for columns that are continuous over two floors can be determined by examination of the figures in S16 Annex F. Item (b) shows a column hinged at the top and fixed at the bottom and the recommended K for this is 0.80, while Situation (c) shows a column pinned at both top and bottom with a recommended K of 1.0. A typical two-storey column will have pinned conditions at the top, continuity at the mid-level and is assumed pinned at the base plate and while it might be argued that K could be less than 1.0 a value of 1.0 is typically used in this situation.

9.2 TORSIONAL BUCKLING OF CRUCIFORM COLUMNS

Columns that are stiff about the two principal axes but are weak about their polar axis may buckle torsionally. Examples of shapes that are in this category include cruciform-shaped columns as shown in Figure 9.2. This type of buckling is covered in S16 Clause 13.3.2. There are three equations, depending on the shape of the section. There is one equation for sections symmetric about both axis, one equation for sections symmetric about one axis and one equation that involves solving a cubic for sections that are not symmetric about even one axis.

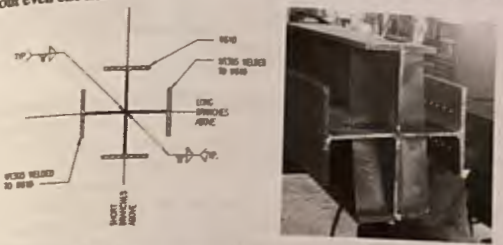


Figure 9.2. Cruciform column made from a wide flange and two tee sections

For symmetric about two axes the equation for torsional buckling is:

$$F_e = \left[\frac{\pi^2 EC_w}{(K_z L_z)^2} + GJ\right]\frac{1}{A * r_o^2} \qquad \lambda e = \sqrt{\frac{F_y}{F_e}} \qquad K_z = \text{torsion effective length factor} = 1.0$$

It can be shown that as $r_o^2 = x_o^2 + y_o^2 + r_x^2 + r_y^2$ where x_o and y_o are the offsets of the shear centre from the centroid of the cross-section. In a doubly-symmetric section both x_o and y_o will be zero. Also as $r^2 = I/A$ we get:

$$F_e = \left[\frac{\pi^2 EC_w}{(K_z L_z)^2} + GJ\right]\frac{1}{I_x + I_y}$$

The axial resistance of the column is then computed from translating the buckling load to a factored resistance in the same way that we handle columns that buckle laterally.

$$C_r = \phi A F_y (1 + \lambda^{2n})^{\frac{1}{n}}$$

Additional information and a theoretical background on torsional buckling can be found in Timoshenko and Gere, *Theory of Elastic Stability*.

EXAMPLE—DESIGN A CRUCIFORM COLUMN

Consider a W610x174 Cruciform column with an unsupported length L_u of 6m in 350W steel. Determine the factored compression resistance C_r taking account of torsional buckling and compare to the C_r from lateral buckling.

The section properties are as follows:

$$J = 2 \text{ (J for W610x174)} = 2\ (2800 \times 10^3 mm^4) = 5600 \times 10^3 mm^4$$

$$C_w = 2 \text{ (Cw for W610x174)} = 2(10{,}900 \times 10^9 mm^6) = 21{,}800 \times 10^9 mm^6$$

$$A = 2(\text{A for W610x174}) = 2(22{,}200 mm^2) = 44{,}400 mm^2$$

$$I_x = I_x \text{ for W610x174} + I_y \text{ for W610x174}$$

$I_x = 1470 \times 10^6 mm^4 + 124 \times 10^6 mm^4 = 1594 \times 10^6 mm^4$

I_y = As above = $1594 \times 10^6 mm^4$

E = 200,000MPa = $200 kN/mm^2$

G = 77,000MPa = $77 kN/mm^2$

F_y = 350MPa = $0.350 kN/mm^2$

$$F_e = \left[\frac{\pi^2 * 200 kN/mm^2 * 21800 \times 10^9 mm^6}{(1.0 * 6000 mm)^2} + 77\frac{kN}{mm^2} * 5600 \times 10^3 mm^4\right] \frac{1}{1594 \times 10^6 mm^4 + 1594 \times 10^6 mm^4}$$

$$F_e = (195 \times 10^4 \frac{kN}{mm^2} + 431 \times 10^6 \frac{kN}{mm^2}) * \frac{1}{3188 \times 10^6 mm^4} = 0.510\frac{kN}{mm^2}$$

$$\lambda e = \sqrt{\frac{F_y}{F_e}} = \sqrt{\frac{0.35 kN/mm^2}{0.510 kN/mm^2}} = 0.828$$

$C_r = \phi A F_y (1 + \lambda^{2n})^{-\frac{1}{n}}$ = Where n = 1.34

$C_r = 0.9 * 44400 mm^2 * 0.350 kN/mm^2 * (1 + 0.828^{(2*1.34)})^{(-1/1.34)}$

$C_r = 13986 kN * (1.603)^{-.746} = 13986 kN * 0.703 = 9835 kN$

Compare to traditional lateral buckling:

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{1594 \times 10^6 mm^4}{44,400 mm^2}} = 189 mm \quad \frac{KL}{r} = \frac{6000}{189} = 31.7$$

From *CISC Handbook* table of Unit Factored Compressive resistances: $\frac{C_r}{A} = 293 MPa$

$C_r = 293 \times 10^3 kN/m^2 * 44{,}000 \times 10^{-6} m^2 = 12{,}892 kN$

In this case, the buckling in the torsional mode occurs at an axial value of 76% of the axial value for lateral mode buckling.

9.3 CALCULATION OF C_r FOR SITUATIONS WHERE THE BUCKLING LENGTH IS DIFFICULT TO DETERMINE

There will be some situations where C_r is desired but the calculation of λ or KL_u is difficult to determine but C_e is known. Take, for example, the curved frame of the cofferdam shown in Figure 9.3. The desire was to have circular beams at the exterior of the frame carrying the pressure of the water, leaving the region inside these beams clear for working once the cofferdam was in place. With the water pumped out of the cofferdam, the exterior pressure of the water puts large compression forces in the curved ring beams. While the unbraced length in the weak axis of the beam was determined by the bracing in the exterior frame of the ring beams, the determination of the unbraced length in the plane of the ring was more difficult to determine.

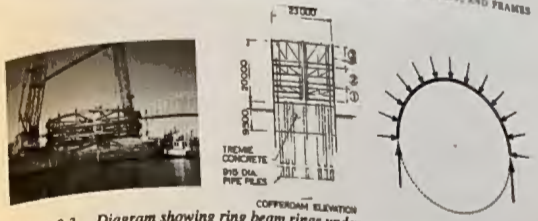


Figure 9.3. Diagram showing ring beam rings under compression and the cofferdam

Resorting to Timoshenko's *Theory of Elastic Stability* the theoretical in-plane buckling value for the ring under compression can be determined. With C_e known, C_r/A can also be determined. Both C_r/A and C_f/A are shown in Figure 9.4, and as the stress at buckling is known, the axial stress resistance in the element permitted by S16 can be found and hence the value of C_r can be determined. Resistance to in-plane buckling of the ring beams is illustrated in Figure 9.5.

In the case examined, the in-plane buckling of the ring beam was a more severe situation than the out-of-plane buckling of the beams, and spokes were added to the cofferdam rings to prevent the in-plane buckling, as shown in Figure 9.6.

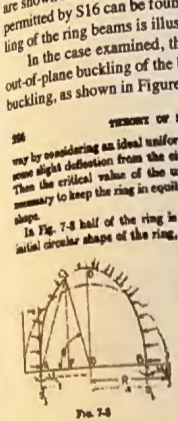


Figure 9.4. C_e can be determined for the in-plane buckling of a ring under axial load by using formulas found in Timoshenko's Theory of Elastic Stability

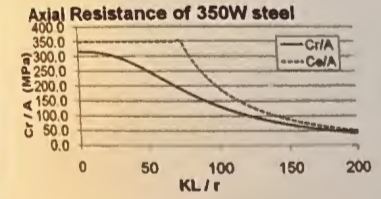


Figure 9.5. With C_r/A known the effective KL/r can be determined and C_r/A can be obtained for the in-plane buckling of the ring beams

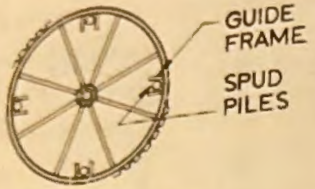


Figure 9.6. An evaluation showed that the rings were overstressed and spokes were added to prevent in-plane buckling

9.4 COLUMN BASE PLATES

The design of base plates for wide-flange columns is covered in S16 Clause 25 and Part 4 of the *CISC Handbook*. The procedure is as follows:

1) Determine the value of factored axial force C_f for the column and size the column.
2) Determine the required base plate area. The base plate area is conservatively determined to be a minimum of $A_{min} = C_f / (0.85 * (\emptyset_c = 0.65) * F'_c)$. A conservative approach governs here as there are some formulas in the concrete code that allow greater contact pressure under certain circumstances. Read A23.3 if you wish to exploit this more.
3) Select the dimensions of the base plate. The base plate should be square where possible as there is less chance of getting anchor rods or base plate orientation wrong. This is illustrated in Figure 9.7.

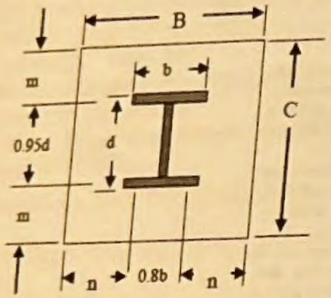


Figure 9.7. Configuration for computing overhangs for base plate calculations
Source: Figure from Part 4 of *CISC Handbook*

4) Compute the required thickness of the base plate. Using the dimensions shown in Figure 9.7, the plate thickness will work out as follows:

$$t_p = \sqrt{\frac{2C_f(Cantilever\_Length)^2}{B*C*\emptyset*F_y}}$$

where (Cantilever_Length) = the larger of m or n

(Alternatively, use the cantilever length from Figure 9.7, assuming uniform pressure on the underside of the base plate and determining the base plate thickness from first principles. Note that this method is easier to remember and will be the way that the remainder of the base plate problems in this text are solved.)

5) Add anchor rods (previously called anchor bolts) for erection and shear resistance. Where possible use four anchor rods as this makes erection of the columns easier. S16 now requires that base plates have four anchor rods, unless steps are taken during construction to ensure safety.
6) Apply a continuous fillet weld between the column and the base plate. This gives even bearing between the column and the base plate and for HSS columns provides the added benefit of providing a seal weld. In the absence of overturning moments on the column, the weld size should be based on the minimum weld size required for the thicker piece of steel being joined. This can be found in Part 6 of the *CISC Handbook* and is as shown in Table 9.4.

Table 9.4. Minimum weld sizes

Material thickness of thicker part joined	Minimum size fillet weld (mm)
$t \le 6mm$	3mm
$6mm < t \le 12mm$	5mm
$12mm < t \le 20mm$	6mm
$> 20mm$	8mm

In general 3mm welds are rare and 5mm or even 6mm will usually be substituted.

EXAMPLE—DESIGN A BASE PLATE FOR THE STEEL HSS COLUMN

Design a base plate for the steel HSS column selected in the previous example. Footing concrete is 25MPa. (C_f = 401kN, HSS 178x178x6.4)

Base plate for HSS 178x178 column with axial load, P_f = 401kN.

$$\text{Min. base plate area} = \frac{C_f}{0.85*(\phi_c = 0.65)*F'_c} = \frac{401kN*10^6 mm^2/m^2}{0.85*0.65*25{,}000kN/m^2} = 29031mm^2$$

Side dimension of square plate = $\sqrt{29031mm^2}$ = 170mm. To satisfy concrete bearing requirements, base plate could be 170mm x 170mm; however, this is less than the size of the HSS column we are using. Increase the base plate by 75mm on all of the four sides to allow for placement of anchor rods and weld between base plate and HSS. Base plate = 75mm + 178mm + 75mm = 328mm say 330mm square. The overhang cantilever is the distance from the centre of the HSS wall to the edge of the base plate.

$$\text{Overhang} = \frac{330mm - 178mm + 6.4mm}{2} = 79mm$$

Option 1:
Use formula in *CISC Handbook* $= t_p = \sqrt{\frac{2C_f(cantilever\_Length)^2}{B*C*\emptyset*F_y}}$

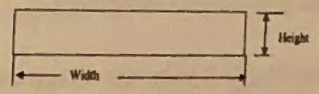
$$t_p = \sqrt{\frac{2*(405kN)*(79mm)^2}{330mm*330mm*0.9*0.300kN/mm^2}} = 13.0mm$$

This would probably be rounded to ½-inch plate (12.7mm)

Option 2:
Work out from first principles

Factored pressure between concrete and plate = 405kN / (.330m x .330m) = 3682kN/m²
Factored Base Plate Moment = Factored Pressure * Width of plate * (Overhang)² / 2
M_f = 3682kN/m² * 0.330m * (0.079m)² / 2 = 3.792kNm
Moment Resistance of plate = $M_r = \emptyset * F_y * Z$

For a plate $Z = \frac{Width * Height^2}{4}$



Z for a rectangle can be found in Part 6 of the *CISC Handbook* but this formula is used so often that it must be memorized just as we have to memorize the moment of a uniformly distributed load on a simple span beam without reference to a table.

$M_r = (\emptyset = 0.9) * (F_y = 300000kN/m^2) * (B = 0.330m) * (t)^2 / 4 = 3.792kNm$

Solve for t

t = 0.013m

Use 1/2-inch plate = 12.7mm (reassuringly same as the formula calculation given in the *CISC Handbook*)

Base plate thickness governs weld size, therefore use continuous 6mm fillet weld between HSS and base plate.

Result:
Use a 330 x 330 x 12.7mm base plate with four 19mm (3/4-inch) diameter A307 anchor rods and a continuous 6mm fillet between the HSS and the base plate.

EXAMPLE—DESIGN A BASE PLATE FOR THE STEEL WIDE-FLANGE COLUMN

Design a base plate for the steel wide-flange column selected in earlier example with W200x46 column having axial load, C_f = 405kN.

Required base plate size to satisfy concrete bearing requirements will be unchanged from previous example (170mm x 170mm minimum)

W200x46 has a flange width of 203mm and a depth of 203mm. Try 350x350 base plate with 4 anchor rods at 40mm off the corners.

$$\text{Overhang} = \frac{350 - 0.95*(203)}{2} = 79mm$$

(The overhang is equal to the plate dimension parallel to web – 0.95 * Section Height) / 2

$$\text{Overhang} = \frac{350 - 0.8(203)}{2} = 94mm$$

(The overhang is equal to the plate dimension parallel to flange – 0.8 * Flange Width) / 2

Overhang of 94mm governs

Contact pressure = 405kN / (0.35m * 0.35m) = 1146kN/m^2

Moment = 1146kN/m^2 * (0.094m)^2*(0.350m) / 2 = 1.77kNm

$M_r = (\emptyset = 0.9) * (F_y = 300000kN/m^2) * (B = 0.350m) * (t)^2 / 4 = 1.77kNm$

Solve for t, get t = 0.0087m. Use 3/8-inch plate t = 9.5mm

Weld size: Flange Thickness on W200x46 = 11.0mm. Web is thinner. As neither the column nor the base plate has elements thicker than 12mm we can call up a 5mm fillet weld.

Result:
350mm x 350mm x 9.5mm base plate with four 3/4-inch diameter A307 anchor rods. Use 5mm continuous fillet weld between column and base plate.

9.5 ANCHOR RODS FOR COLUMN BASES

Anchor rods are installed in column base plates for several reasons:

1) To assist with erecting the column by holding it in place prior to the attachment of beams. S16 requires that there be a minimum number of 4 anchor rods on column bases but allows for fewer anchor rods if "special provisions" are taken.
2) To provide stability to the column base to make sure that it is restrained. The forces involved will generally be small and a resistance of 2% of the column load will be sufficient. Use of four 3/4-inch diameter A307 anchor rods will provide a shear resistance of 4(33.4kN) = 134kN. This is sufficient shear capacity to resist an axial load of 134kN / 0.02 = 6680kN which will adequately cover all columns of low-rise buildings.

3) To resist any minor shears occurring due to minor unbalanced loads, wind load on the column or cladding attached to the column.
4) To resist wind uplift loads.
5) To resist significant overturning moments and shears from the column being part of a moment frame.

Unless resisting large tension or overturning loads, anchor rods are generally constructed from A307 anchor rod material. This is a low-grade bolt material with a minimum tensile strength F_u of 414MPa. Bolt data, including available sizes, is listed in *CISC Handbook* Table 3-1. Anchor rods should have a minimum of 40 diameters embedment (760mm for a 3/4-inch diameter bolt) or be hooked to provide pull-out resistance. An alternate method of providing resistance to pull-out is to thread the end of the embedded anchor rod and screw on a nut and washer or bearing plate.

For larger loads, the Dywidag bar makes an excellent anchor rod for the following reasons:

1) The Dywidag bar has an upset thread so that no reduction is necessary for area reduction for threading ($A_n = A_g$)
2) The Dywidag bar has a continuous thread that makes it easier to extend if the bolt is set too low or if the top of the bolt becomes damaged.
3) The Dywidag bar comes in larger diameters than does A307 anchor rods.

By putting a nut and a square washer plate on the embedded end, anchorage can be easily provided if the straight length embedded in the concrete is too short to provide anchorage

For base plates where there is significant moment (e.g., for high signs or moment frames), consider adding vertical plates to directly transfer shear to the anchor rods. An example of a moment-frame column where this has been done is shown in Figure 9.8.

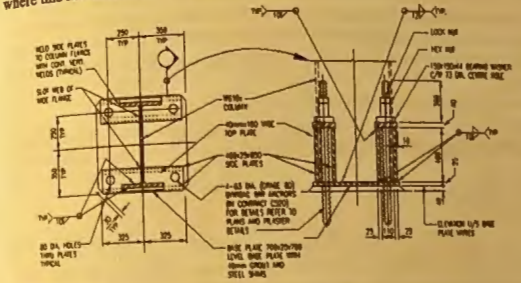


Figure 9.8 Moment-frame column base

Another example of moment-frame column base is shown in Figure 9.9.



Figure 9.9. Moment-frame base column, compression forces and tension forces

Compression forces are shown in red and the tension forces in green. The aim is to transfer the forces from the tension flange to the anchor rod and from the compression flange to the concrete. Both of these transfers involve an eccentricity that has to be resolved. Using a traditional flat base plate to transfer these forces very quickly results in very thick base plates that lead to difficult welds. Adding blocks on the side as shown in Figure 9.9 means there is a good lever arm for the moment (see the white arrows) and a good length of vertical plate to carry the shear.

9.6 CALLING UP COLUMNS ON STRUCTURAL DRAWINGS

Sufficient information must be shown on the structural drawings to put across the structural intent of the designer. For columns this includes the following information:

- The location of the column including any offsets from gridlines must be shown on plan. (S16 Clause 4.2.1 requires this.)
- The size of the column (e.g., W200x27 or HSS127x127x6.4).
- The orientation of the column if the column is
  - A wide-flange column
  - A rectangular HSS column
  - At an angle to the gridlines
- The details at the top of the column, including
  - Cap plate details for HSS columns
  - Stiffeners in beams if they cantilever over the column.
- Column splice details if it is intended that the column will be more than a single lift.
- The base plate information, including:
  - Size and thickness of base plate
  - Weld between the column and base plate
  - Number, size and grade of anchor rods
  - Dimensions to anchor rods from the column centreline. This is usually covered in details on the base plates.
  - Embedment on anchor rods is usually shown on drawings.

Two examples of base plate details are shown in Figure 9.10.

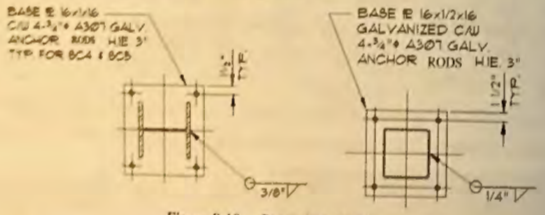


Figure 9.10. Base plate details

9.7 COMBINING TENSION AND MOMENT

Axial tension and bending is covered in S16 Clause 13.9, which requires evaluation of two equations:

1) $\frac{T_f}{T_r} + \frac{M_f}{M_r} \le 1$

where M_r is $\emptyset F_y Z$ for Class 1 and 2 section and $\emptyset F_y S$ for Class 3 and 4 sections.

2) $\frac{M_f}{M_r} - \frac{T_f Z}{M_r A} \le 1$ For Class 1 and 2 sections and

$\frac{M_f}{M_r} - \frac{T_f S}{M_r A} \le 1$ For Class 3 and 4 sections

(Equation 2 is only required if the member is unsupported, in which case use M_r' in Equation 2, taking account of the unsupported length of the member.)

EXAMPLE—DESIGNING TENSION AND MOMENT

Determine if a Class C G40.21 102x102x6.4 HSS with $F_y = 350MPa$ with a 22mm diameter hole through two of the walls of the tube can support a tension of 200kN and a bending moment of 15kNm. The moment is oriented such that the holes are in "webs" not "flanges." Ignore the connection design and related shear lag issues at this stage.

Solution:

The easiest solution to this problem is to look in the column capacity tables (green pages) of *CISC Handbook* and find that for an HSS 102x102x6.4 column the moment resistance is 25.6kNm An alternative would be to find M_r using $\emptyset F_y Z$ for Class 1 and 2 section and $\emptyset F_y S$ for Class 3 and 4 sections. We have computed T_r to be 702kN in Chapter 5.

$$\frac{T_f}{T_r} + \frac{M_f}{M_r} \le 1 = \frac{200kN}{702kN} + \frac{15kNm}{25.6kNm} = 0.29 + 0.59 = 0.87 (OK \le 1)$$

As the interaction ratio is less than 1.0 the section works under the combined tension and bending.

S16 does not give a formula for tension combined with bending about two axes, but use of a combination such as the one that follows would be appropriate unless the beam was unsupported. In that case, modifications by use of M_r' instead of M_r would be appropriate.

$$\frac{T_f}{T_r} + \frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \le 1$$

Tension members with bending are less common than compression members with bending. An example of a tension member with bending would be the bottom chord of a truss where there is a hanging load in the middle of the panel.

9.8 BEAM-COLUMNS

Beam-columns, especially those that are part of the lateral frame of the building, pose more difficulty in determining their resistance than do columns that are carrying primarily axial load. For a beam-column with axial force and moment about the strong axis only, the flowchart shown in Figure 9.11 defines the procedure.

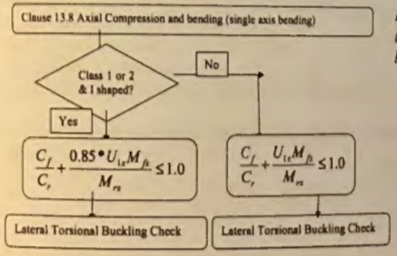


Figure 9.11. Flowchart to calculate axial load of beam-columns

For braced frames: $U_1 = \left[\frac{\omega_1}{1-\frac{C_f}{C_r}}\right] \geq 1$ (See flowchart)

For unbraced frames $U_1 = 1.0$

C_r = Axial resistance usually calculated from column capacity tables (green pages) of *CISC Handbook*, which takes account of weak axis buckling. Use K = 1.0 if P-Delta effects have been included in the analysis.

M_{rx} = Moment resistance about the same axis as the applied moment taking account of unsupported length.

$$C_e = \frac{\pi^2 * E * I_{strong}}{L^2}$$

Bending about two axes makes it more necessary to check the cross-sectional strength check and the overall strength check as well as the lateral torsional buckling. The flowchart for bending about two axes is shown in Figure 9.12.

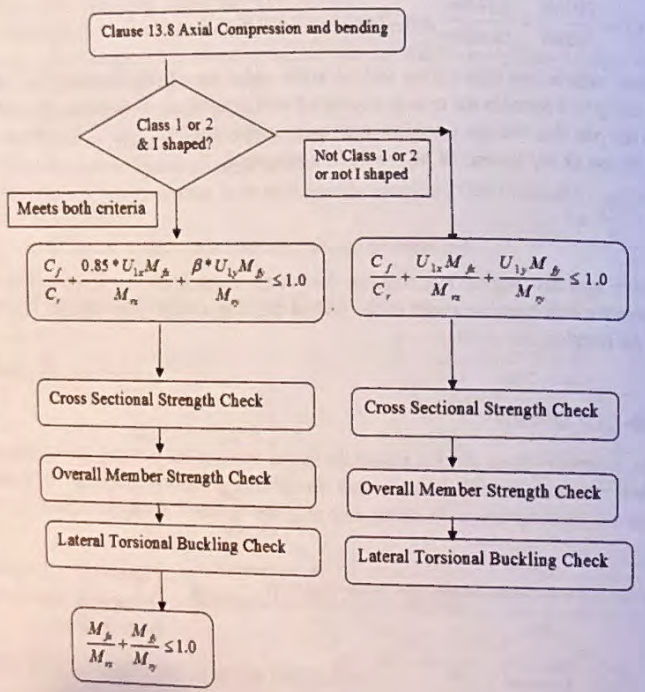


Figure 9.12. Flowchart for bending about two axes

Three aspects of this procedure should be noted:

a) Note that to use the 0.85 factor the section shape must be I-shaped and a Class 1 or 2 section. HSS sections do not get to use the 0.85 factor.

b) Prior to S16-01 the β factor was a constant value of 0.6. Now:

$$\beta = 0.6 + 0.4 * \lambda_y \leq 0.85$$

c) The overall member strength check and the lateral torsional buckling strength check use U_1 values of 1 for unbraced frames and allow U_1 values to be calculated in unbraced frames. U_1 is frequently less than 1 in unbraced frames as ω is less than 1 with the moment reversals that occur in moment-frame columns.

The analysis of the moment-axial strength checks for bending about two axis is lengthy and complex and it is easy to make mistakes. Use a canned program such as S-Steel® (make sure you are using a version that uses S16-09 rather than S16-01 or S16.1-94 as S-Steel® has evolved over several versions of the steel standard) or write your own program check against sample problems before solving the problem you are really interested in. Refer to the example in Part 4 of the *CISC Handbook* for a numerical example involving bending about two axis and check whichever program you are using against a known solution.

9.9 COLUMNS IN BRACED AND UNBRACED FRAMES

In S16 the interaction formulas set U_1 to 1 in unbraced frames. The situation in braced frames is more complex.

In the unbraced frame U_1 can be set to 1 as the maximum column moment occurs at the column ends and the moments should be amplified in the analysis of the structure to include the effects of P-Delta and notional loads. Moment frames require this type of computer-assisted analysis. Analysis by hand of a laterally loaded moment frame is no longer considered a cost-effective method of analysis in a design office setting, as the P-Delta effects and notional load effects on the frame cannot be included in a reasonable time period.

In braced frames U_1 is amplified and is more often 1 or greater, as the moment can be greater in the midspan in a way that is not reflected in a P-Delta analysis of the structure. For example, columns acting as wind girts in a braced frame have their midspan moment increased by the axial effect and this amplification occurs only in the member length. Axial amplification of the moments within a member is not picked up by most frame analysis programs or occurs in individual members (such as the wind girt) where a P-Delta analysis is not undertaken.

The qualities of unbraced frames (moment frames) and braced frames can be compared and contrasted as follows:

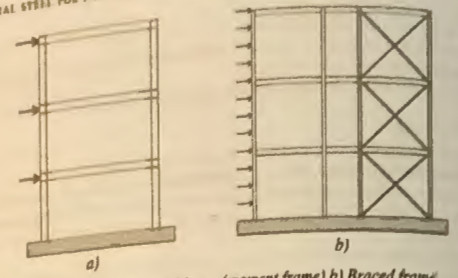
Unbraced frame (moment frame):

- Maximum moments at beam /column joints
- Get forces on column from computer analysis that includes P-Delta and notional loads

Braced frame:

- Maximum moments can occur within column span
- Evaluate column individually and apply U_1 factor
- U_1 will generally be 1.0 or greater
- Notional loads will not have much effect on the columns but will increase forces on braces

These qualities are illustrated in the sample unbraced frames (moment frame) shown in Figure 9.13 and the sample braced frame shown in Figure 9.14.



a) b)

Figure 9.13. a) Unbraced frame (moment frame) b) Braced frame

U_1 is a term to represent the magnification of the moment by the axial force. U_1 increases the moment to account for secondary bending necessary to keep the axial load flowing through the deformed beam. The moment magnification increases as we approach the buckling axial load for the column. The flowchart to calculate U_1 is shown in Figure 9.15.

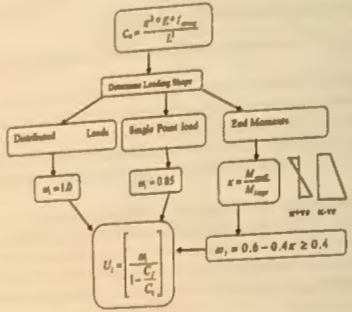


Figure 9.15. Flowchart to calculate U_1

Note that in the case of end moments, κ can vary from -1.0 to 1.0, so the value of ω_1 can vary from 0.4 to 1.0.

However if there are distributed loads in the span of the beam-column $\omega = 1$ and the equation simplifies to:

$$U_1 = \frac{1}{1 - \frac{C_f}{C_e}}$$

where C_e is the Euler buckling strength about the axis of concern $C_e = \frac{\pi^2 * E * I}{(KL)^2}$ if we are bending about the stiff axis of a wide-flange use the stiff axis moment of inertia when determining C_e.

There are several situations where we would have axial compression combined with bending. One example would be the top chord of a truss where joists or deck is supported between panel points in the truss. Another example is a column that supports a lateral load from wind on the exterior cladding.

To summarize:
The U_1 term covers the axial amplification of the moment as shown in Figure 9.16.

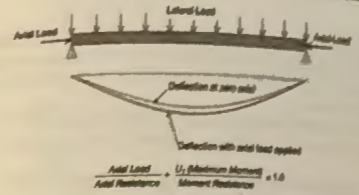


Figure 9.16. Axial amplification of the moment

9.10 DESIGN OF BEAM-COLUMNS

In general, the designer must check the following three potential limit states (in addition to preventing local buckling):

1) cross-sectional strength;
2) overall (in-plane) member strength; and
3) lateral-torsional buckling strength (out-of-plane).

Exceeding the capacity of the beam-column for any of these limit states implies failure of the member.

A single equation in S16 is used for all three limit states in design, the difference being in the interpretation of the variables. The beam-column design interaction equation for *Class 1 and 2 I-shaped members* is:

$$\frac{C_f}{C_r} + \frac{0.85U_1M_{fx}}{M_{rx}} + \frac{\beta U_1 M_{fy}}{M_{ry}} \leq 1.0 \text{ and } \frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \leq 1.0 \text{ [Clause 13.8.2]}$$

where the subscripts x and y indicate the "strong" and "weak" axis cases, respectively, C_f and M_f are the factored compressive force and the maximum factored moment (including P-Δ effects and notional load), respectively, in the beam-column, and:

$$\beta = 0.6 + 0.4 * \lambda_y \leq 0.85 \text{ [Clause 13.8.2]}$$

where λ_y is the weak axis slenderness ratio (same as the one used for columns). Although not stated explicitly in S16, use the same K for λ_y in determining β as for C_{ry} (typically K = 1.0).

Since the definitions of the other parameters vary depending on the limit state being checked, they are defined in the following subsections.

Limit State 1—Cross-sectional Strength

This limit state only needs to be checked for members in *braced frames* simply because for unbraced frames it can never govern. For this limit state (only), M_{fx} and M_{fy}, if both are present, coexist at the same cross-section (i.e., they are not necessarily the maximum values on the member). The various as-yet-undefined parameters are defined as follows:

- C_r is as defined in Clause 13.3 with $\lambda = 0$;
- M_r is as defined in Clause 13.5;
- U_1 is as defined in Clause 13.8.4, but not less than 1.0; and
- λ_y is taken as 0 in the calculation of β.

Limit State 2—Overall Member Strength

This limit state addresses the inplane (i.e., no LTB) behaviour of the member as a whole, accounting for the Pd effect. It is analogous to flexural buckling of a column, but the effect is influenced by the presence of bending moments. The parameters in the design equation that were defined in the previous section are *redefined* for this limit state as follows:

- C_r is as defined in Clause 13.3 (i.e., use $C_{r,min}$), except that for *uniaxial* bending cases, the axial capacity related to buckling *about the bending axis* is used (even if it does not give the lower capacity). Use K = 1.0 [Clause 10.3.2];
- M_r is as defined in Clause 13.5;
- U_1 is as defined in Clause 13.8.4 for members in *braced frames*; and
- U_1 = 1.0 for members in *unbraced frames*.

Limit State 3—Lateral-Torsional Buckling Strength

Of course, this limit state need only be checked if no bracing is provided that would prevent LTB. Also, LTB cannot occur if the bending is about the weak axis only. The parameters in the design equation are *redefined* yet again for this limit state as follows:

- C_r is as defined in Clause 13.3 and is based on *weak axis* buckling using the actual weak axis effective length (£ 1.0) [Clause 10.3.3], although it is common to use K = 1.0 for simplicity;
- M_{rx} is as defined in Clause 13.6;
- M_{ry} is as defined in Clause 13.5;
- U_{1x} is as defined in Clause 13.8.4, but not less than 1.0, for members in *braced frames*;
- U_{1y} is as defined in Clause 13.8.4 for members in *braced frames*; and
- U_1 = 1.0 for members in *unbraced frames*.

One thing that is useful to note in the interest of design efficiency is that for cases where LTB is possible, Limit State 3 will always govern unless one of the following situations applies:

1) You have either uniaxial weak-axis bending or biaxial bending and $\omega_{1y} < 1.0$ (Limit State 1 could govern for members in braced frames);
2) You have either uniaxial strong-axis bending or biaxial bending and $C_{rx} < C_{ry}$ due to the presence of intermediate lateral braces (Limit State 2 could govern); and/or
3) You take K < 1.0 in determining C_{ry} for Limit State 3 (either Limit State 1 or Limit State 2 could govern).

Even if one or more of these cases applies, where LTB is possible it is still common for Limit State 3 to govern the design of beam-columns.

EXAMPLE—CHECK THE INTERACTION OF THE BEAM-COLUMN

Check the interaction of the beam-column shown in Figure 9.17 using the interaction formula and method above. Try W310x118. Use 345W steel. Assume braced frame.

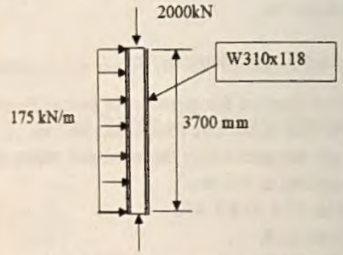


Figure 9.17. Sample beam-column

Solution:

The section should first be checked to determine Class of section.

Table 5-1 of the *CISC Handbook* shows this to be a Class 2 section for bending.
Table 4-3 of the *CISC Handbook* shows this to have a Class 1 Web and Class 2 flanges.

Section is Class 2 for beam-column and the section is I-shaped.

C_f = 2000kN For W310x118 with kL = 3750mm C_r = 3830kN

$M_f = 175 * 3.7^2/8 = 300\text{kNm}$

For W310x118 L_u = 4920mm (OK >3750mm)

M_{rx}(kNm) (L<L_u) = 606kNm

$$C_e = \frac{\pi^2 * E * I}{(KL)^2} = \frac{\pi^2 * 200 * 10^6 kN/m^2 * 275x10^{-6} m^4}{(3.7m)^2} = 39651kN$$

(stiff I used, as bending is in the plane of stiff I)

Have interior loads in span so $\omega_1 = 1.0$

$$U_1 = \frac{1}{1-\frac{C_f}{C_e}} = \frac{1}{1-\frac{2000kN}{39651kN}} = 1.05$$

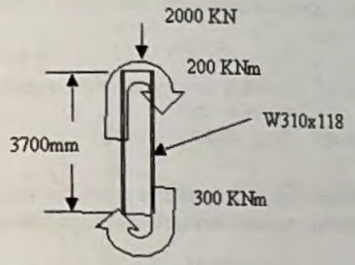
Place the above information into the interaction formula:

$$\frac{C_f}{C_r} + \frac{0.85 * U_1 * M_f}{Mr} = \frac{2000kN}{3830kN} + \frac{0.85 * 1.05 * 300kNm}{605kNm} = 0.522 + 0.443 = 0.963$$

(OK: <1.0)

EXAMPLE—DESIGN COLUMN IN MOMENT FRAME

Column in moment frame (*not a braced frame*) with axial force and end moments. P-Delta effects have been included in the analysis as have notional loads to get the forces shown. Perform the lateral torsional buckling check.



Lateral Torsional Buckling Strength Check:

- C_r is computed using K = 1.0 provided that P-Delta effects have been included when computing the end moments on the column. C_r is determined using the minimum buckling resistance; it can therefore be chosen from the tables. C_r for the W310x118 with an L_u of 3700 is extracted from the column capacity tables (green pages) of the *CISC Handbook* and interpolation is used to get between C_r = 3830kN at L_u = 3750mm and C_r = 3940 at L_u = 3500mm.
- M_r is based on a laterally unsupported member. From the beam resistance tables given in the beam selection tables (blue pages) of the *CISC Handbook* the L_u for a W310x118 is 4920mm and any unsupported lengths less than this have an M_r of 603kNm. With our unsupported length of 3700mm we therefore have a M_r of 605kNm. The value of L_u can also be found in the column capacity tables (green pages) of the *CISC Handbook* with the factored axial compressive resistance.
- U_1 for beam-columns in unbraced frames is 1.0.

$$Interaction = \frac{2000}{3850} + \frac{0.85*1.0*300}{605} = 0.519 + 0.421 = 0.94 \le 1.0\,(OK)$$

In general for building columns the lateral torsional buckling check will govern for uniaxial bending, but cross-sectional strength check and overall member strength check should be considered if the situation is strange.

EXAMPLE—CHECK CROSS-SECTIONAL AND OVERALL MEMBER STRENGTHS

Continue the previous example and perform the cross-sectional strength check and the overall member strength check.

Cross-sectional strength check: (shown for completeness but will not govern in unbraced frames)

- C_r is computed using $\lambda = 0$ giving $C_r = \emptyset AF_y$
- $C_r = (0.9) * (15{,}000 \text{ x } 10^{-6}mm^2) * (345{,}000kN/m^2) = 4658kN$
- M_r is determined as though the section is laterally supported.
- U_1 is taken as 1.0

The interaction for cross-sectional strength check is:

$$\frac{2000}{4658} + \frac{0.85*1.0*300}{605} = 0.429 + 0.421 = 0.85 < 1.0\,(OK)$$

Overall Member strength check:

- C_r is computed using K = 1.0 provided that P-Delta effects have been included when computing the end moments on the column. For uniaxial bending $C_r = C_{rx}$ (Strong axis bending).
- Use strong axis bending. $r_{strong} = 136mm$
- KL/r = 1.0 * 3700 / 136 = 27.2
- $C_r/A = 297MPa$ (use Table 4-4 of the *CISC Handbook*)
- $C_r = 297{,}000kN/m * 15{,}000 \text{ x } 10^{-6}m^2 = 4455kN$
- M_r is based on a laterally supported member
- $U_1 = 1.0$ for unbraced frame

In the example in Part 4 of the *CISC Handbook* the interaction for overall member strength check is:

$$\frac{2000}{4455} + \frac{0.85*1.0*300}{605} = 0.449 + 0.421 = 0.87 < 1.0\,(OK)$$

As expected (with the column bending about one axis only), the values of 0.85 for cross-sectional strength check and 0.87 for overall member strength check are below the 0.94 for lateral torsional strength check.

9.11 SEISMIC CONSIDERATIONS FOR COLUMN DESIGN

For columns that are in multi-storey buildings that derive their resistance by brace bay frames we are required by S16 Clause 27.5.5.2 to have columns that are not Class 4 sections – they therefore need to meet the requirements of Class 3 sections.

As we saw from Chapter 3 on section class for square HSS to meet Class 3: $\frac{b}{t} \le \frac{670}{\sqrt{F_y}} = 35.81$

Looking at the b-over-t ratios for rectangular sections as given in Table 9.5, we see only two sections that do not comply, namely HSS 254x254x6.4 and HSS 305x305x6.4.

Table 9.5. b-over-t ratios for rectangular HSS sections (G40.21)

b is taken as outside dimension minus 4 times wall thickness

	<-----Outside Dimension of the HSS----->											
Wall	38	51	64	76	89	102	127	152	178	203	254	305
3.18	7.9	12.0	16.1	19.9	24.0	28.1						
3.81	6.0	9.4	12.8	15.9	19.4	22.8						
4.78	3.9	6.7	9.4	11.9	14.6	17.3	22.6	27.8	33.2			
6.35		4.0	6.1	8.0	10.0	12.1	16.0	19.9	24.0	28.0	36.0	44.0
7.95				5.6	7.2	8.8	12.0	15.1	18.4	21.5	27.9	34.4
9.53				4.0	5.3	6.7	9.3	11.9	14.7	17.3	22.7	28.0
11.13							7.4	9.7	12.0	14.2	18.8	23.4
12.70							6.0	8.0	10.0	12.0	16.0	20.0

9.12 P-DELTA EFFECTS

P-Delta effects are usually more of a problem with structures that derive their stability from the bending stiffness of members (moment frames) than those structures that rely on axial stiffness of members for stability (braced frames).

There is considerable detail in Appendix J giving calculation methods for computing P-Delta effects. An easier method is to have a computer program perform this task.

EXAMPLE—P-DELTA EFFECTS

P-Delta effects can be illustrated by the simple model shown in Figure 9.18. This might represent a weight supported on a pole under wind loading. As the wind deflects the weight laterally the load becomes eccentric from the base and a moment due to the displaced weight (equal to P*D) results. The eccentricity of the load causes the weight to deflect further and the base moment to increase.

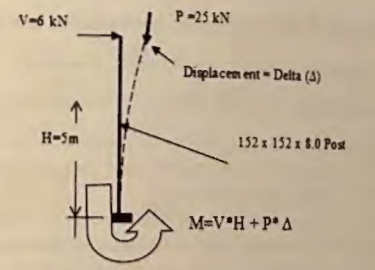


Figure 9.18. Simple model to illustrate P-Delta effects

For the model shown in Figure 9.18, the elastic deflection due to the horizontal load can be computed as follows:

$$\Delta = \frac{V*H^3}{3*EI} = \frac{6kN*(5m)^3}{3*200*15.1} = 0.083m$$

The elastic moment at the base of the post can then be calculated as:

$$M = V*H + P*\Delta = (6kN * 5m) + (25kN * 0.083m)$$
$$= 30kNm + 2.08kNm = 32.1kNm$$

9.14 OVERALL FRAME STABILITY

While much of our design consists of guarding against buckling failure of individual members it is necessary in some cases to examine stability of the overall frame.

Overall frame stability or buckling analysis is different from P-Delta analysis in that it is not necessary to laterally displace the structure to get a buckling failure. This can be illustrated in the example shown in Figure 9.20.

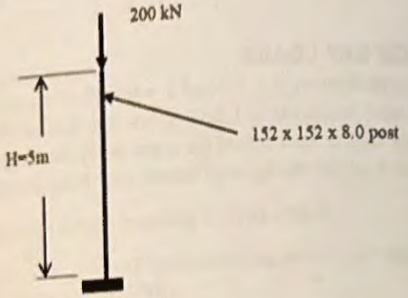


Figure 9.20. Overall frame stability

Using Annex F of S16 we find the theoretical K value is 2.0 with the recommended design value being 2.0. With K being 2.0 the theoretical buckling load would be:

$$P_{cr} = \frac{\pi^2(EI)}{(KL)^2} = \frac{\pi^2(200*15.1)}{(2.0*5m)^2} = 298kN$$

Pframe Screen requesting buckling analysis of this problem and the results are shown in Figure 9.21.

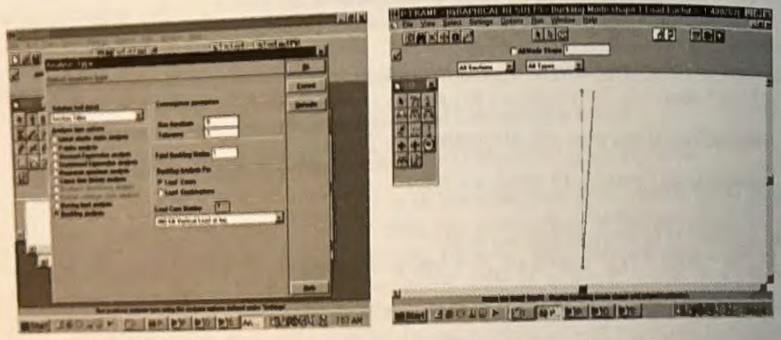


Figure 9.21. Pframe screens during buckling analysis

Results for first buckling mode. Buckling Force = 200kN * 1.49935 = 299.9kN
Compares well with 298kN found by hand analysis.

CHAPTER 10
CONNECTION DESIGN

This chapter does not intend to be a comprehensive manual for connection design; the intent is to provide some background and redirection to source where more information can be found. This chapter also discusses the designer's involvement with a structural steel project after the design is complete, including the review of shop drawings and the site review of the steel erection. Connections between hollow structural section (HSS) members, such as those found in trusses, are discussed in Chapter 6 on trusses. Connections for seismic elements are covered in the chapters on seismic design, but these sections rely on the principles we establish in this section.

10.1 CONNECTION DESIGN FOR THE STRUCTURAL DESIGNER

With the possible exception of seismic elements, in the Canadian structural steel industry it is standard practice that the fabricator designs most of the steel-to-steel and "simple" beam connections, usually with the help of his or her detailer and a connection engineer that the fabricator retains. Nevertheless, the designer needs to know enough about the design of connections to verify that the fabricator has detailed them to meet the intent of the designer. The design engineer must also knowledgeably respond to questions on the connections posed by the fabricator.

The engineer of record is responsible for performing the structural design, including sizing the elements and determining the forces on any connections. The fabricator then is responsible for detailing the connections based on loads the engineer of record provides. However, the lines of responsibility are not clear-cut. If a problem were to occur with a connection and it was proved that the issue had arisen from the work of the fabricator's engineer, the engineer of record would still be implicated, even in those cases where the drawing general notes require the shop drawings to be sealed by a professional engineer. The engineer of record will still be involved with the design of the connections, will review shop drawings, and will at the very least expend time fixing any problems identified. Therefore it is in the best interests of all parties to make certain that the connections satisfy the intent of the structural design.

To avoid questions during the detailing stage, forces for connections other than uniformly loaded simple span beams should be clearly shown on drawings. This includes any "pass through" forces where the connection is required to act as a short compression or tension element. Such an element transfers axial load in the beam on one side of the column to the beam on the other side.

Additionally, connections of seismic bracing should be detailed by the designer and shown clearly on structural drawings. This includes details of anchorages required to transfer forces between the structural steel and supporting concrete elements. Supporting concrete elements can include concrete beams and columns, in the case of a steel penthouse of a concrete building, and concrete foundations for buildings using steel throughout.

Also, because most standard beam-to-girder connections are field-bolted, with any welding occurring in the shop, it is necessary for the designer to have an understanding of bolted connections and welding capacities.

10.2 ADDITIONAL TRAINING IN THE ART OF DETAILING

The material contained in this chapter provides some information on connection design and review from the standpoint of the structural engineer. The aim is to show some of the issues and provide a workable background for some the resources available. A full discussion of connection design requires considerably more training than can be presented here, and those desiring more background should consider options for taking further training in either Canadian or American standards.

For those interested in more training on connection design with Canadian standards, a course is offered that covers only connection design in a 10-unit, 120-hour computer-based-training format. The course uses CSA Standard S16 and the *CISC Handbook of Steel Construction*. Additional information can be found at Connectiondesign.ca.

For those interested in connection design training with American standards, the American Institute of Steel Construction (AISC) has produced a 10-module, free online program that covers the preparation and review of structural steel shop drawings. The Detailer Training Series includes the basic detailing process, bolted and welded connections, various types of shear connections, and vertical and horizontal bracing. The course is in imperial units. More information can be found on the AISC website using the search term "Detailer Training Series."

10.3 BEAM CONNECTION DESIGN FORCE

It is necessary for the fabricator's engineer to know the connection design shear force. The most common connections are those from a beam to either a column or supporting girder, and these connections are designed and detailed to transfer shear force only. The engineer of record will not specify the shear force at the end of each beam and the connection design engineer will not be expected to do a structural analysis to determine the loads on each beam. Therefore, the structural drawings should state that the beam connection is to be designed for a shear corresponding to the uniform load flexural capacity for a laterally supported beam of the same span and section. In graphical form, this can be expressed as shown in Figure 10.1. It should be noted that the beam is connected for the load calculated to give the moment capacity of the beam, and this shear will usually be much less than the shear resistance, V_r, determined from the beam load tables (blue pages) of the *CISC Handbook*.

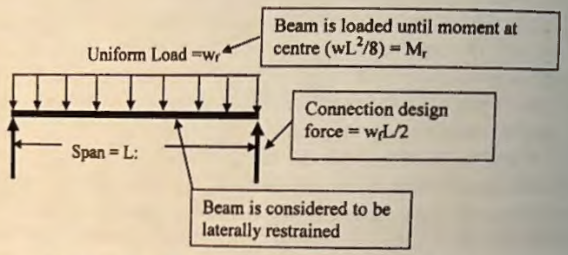


Figure 10.1. Beam connection designed for shear

This shear capacity will be sufficient for most beams, and possibly conservative if the governing design criterion of the beam is deflection or lateral-torsional buckling. (Where this value is considerably conservative and the joint is congested, the actual design shear should be specified.) The capacity will not be conservative if there are point loads applied close to the end of the beam, such as would occur with the transfer of a column. Under such circumstances, the structural drawings should state the connection design force to be used by the fabricator.

There are two ways of getting the factored shear force for connection design:

- Manipulate the equations $V_{end} = wL/2$ and $M_r = M_f = wL^2/8$.
- Use the beam load tables (blue pages) of the *CISC Handbook*.

EXAMPLE—SHEAR CONNECTION FORCE DETERMINATION (UDL)

Determine the connection force for a W410x39 of 350W steel with a 7.5m span.

Solution 1:

Determine the M_r of a laterally supported W410x39 using the procedures outlined in Chapter 3. The easiest method is to look it up in the beam load tables (blue pages) of the *CISC Handbook*, where we find that M_r = 227kNm.

$$\text{The factored moment} = \frac{w_f L^2}{8} = \frac{w_f L}{2} * \frac{L}{4} = M_r$$

$$\text{The end shear} = \frac{w_f L}{2} = \frac{4M_r}{L} = V_f$$

$$\text{Factored connection force} = V_f = \frac{w_f L}{2} = \frac{4M_r}{L} = \frac{4 * 227\,kNm}{7.5m} = 121kN$$

Solution 2:

Use the beam load tables (blue pages) of the *CISC Handbook* and find the "Total Uniformly Distributed Factored Load for Laterally Supported Beams (kN)." For a W410x39 with a span of 7.5m, the total factored load is 242kN (as illustrated in Figure 10.2).

As half of this will be picked up at each end of the beam, the factored connection design shear force = 242kN / 2 = 121kN.

The shear resistance of the beam, V_r, is 480kN and as this shear cannot be developed in a 7.5m long W410x39 before the beam develops a plastic hinge from the moment, it would be unusual to design the connection at the end of the beam for the shear capacity of the beam. Note that the beam load tables take account of both the shear and moment capacities of the beam and therefore the end shear on the W410x39 is limited to 960kN/2 = 480kN corresponding to its shear capacity. The beam load table for the W410x39 section does not give a capacity value with a 1.0m span; however, the load capacity of the beam at a span of 1.0m would also be 960kN so that shear capacity of the beam is not exceeded.

BEAM LOAD TABLES — **CSA G40.21 350W**
W Shapes — **ASTM A992, A572 grade 50**

Total Uniformly Distributed Factored Loads for Laterally Supported Beams (kN)

Designation	W410											Approx. Deflect.
Mass (kg/m)	149	132	114	100	85	74	67	60	54	46	39	(mm)
1 000												1
1 500										1 160	960	1
2 000					1 860	1 640	1 480	1 280	1 240	1 100	907	2
2 500					1 720	1 500	1 350	1 180	1 040	879	725	4
3 000	2 640	2 320	2 000	1 700	1 430	1 250	1 130	985	869	733	604	5
3 500	2 310	2 020	1 750	1 510	1 230	1 070	965	845	745	628	518	7
4 000	2 020	1 770	1 530	1 320	1 070	938	845	739	652	550	453	10
4 500	1 790	1 570	1 360	1 160	955	834	751	657	580	489	403	12
5 000	1 610	1 420	1 220	1 060	859	750	676	591	522	440	363	15
5 500	1 470	1 290	1 110	962	781	682	614	537	474	400	330	18
6 000	1 350	1 180	1 020	882	716	625	563	493	435	366	302	22
6 500	1 240	1 090	940	814	661	577	520	455	401	338	279	26
7 000	1 150	1 010	873	756	614	536	483	422	373	314	259	30
7 500	1 080	944	815	705	573	500	450	394	348	293	242	34
8 000	1 010	885	764	661	537	469	422	369	326	275	227	38

Figure 10.2. Total factored load for a W410x39 beam

10.4 BASIC BOLTED CONNECTION DESIGN

The following are very basic guidelines for the design of steel-bolted connections in beams used in buildings in Canada. It is intended that the information contained in this chapter be read in conjunction with the *CISC Handbook*, where additional information on bolted connections is found.

Bolted connections should use imperial-sized bolts as these bolts are less expensive and more readily available than metric-sized bolts. Even on projects where the drawings are in metric units, the bolts should be called up using imperial sizes such as 3/4-inch (19mm) diameter, not as M20 bolts. When designing connections, bolt capacities should be based on imperial sizes. A 3/4-inch diameter bolt has less capacity than an M20 bolt but has the advantage of being more readily available. Attempting to use M20 bolts will result in the first request for information (RFI) from the fabricator being a request to switch to 3/4-inch (19mm) diameter bolts.

For most connections, including many moment-frame connections, the shear in the beam is transferred by a bolted connection at the web. Connections between the beam web and the supporting member are provided by bolting to a variety of elements, including a single angle connection, a double angle connection, a vertical end plate shop-welded to the supporting girder, and shear plates welded to columns.

It is desirable that connections should not be the weak link in the chain of potential failure, and due to the greater variability in the connection resistance, the $\emptyset_b$ factor for bolts is given as 0.8. (This is an increase from the value of 0.67 in S16 used prior to 2001.)

Beam connections are frequently performed using double angle connections, but a single angle connection is also used sometimes, as shown in Figure 10.3.

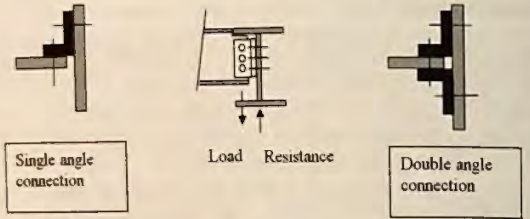


Figure 10.3. Sample beam connections

Many of these connections are illustrated in the yellow pages of the *CISC Handbook* with accompanying tables to assist in design. Care should be taken that the tables are appropriate for the connection being designed (e.g., the single angle connection shown in Figure 10.3a (left) is bolted to both beams, but Table 3-40 of the *CISC Handbook* covers the situation where the angle is bolted to the supported beam and welded to the supporting girder).

Connection design must check for a variety of failure modes, including the following:

- Overstress of the bolts.
- Inability to transfer the shear through the eccentricity from the bolt centreline to the centreline of the supporting member.
- Bearing failure of the plate from trying to pick up too much bolt force on too thin a plate.
- Block tearout of the web or supporting plate.

Three basic types of bolts used in connections are shown in Table 10.1.

Table 10.1. Three bolt types used in connections

A307 Bolts	Typically used for anchor rods between concrete and steel columns. Seldom used for steel to steel connections in structures and may not be used in slip-critical connections
A325 Bolts	High strength bolts used in most structural connections for buildings
A490 Bolts	Higher strength and less ductile than A325 bolts, these are not typically a first choice in building construction due mainly to their higher cost

Bolt holes are generally sized to give some tolerance and for ease of bolt placement and, as per the provisions of S16 Clause 28.4 (and unless explicitly precluded by the contract documents), may be punched, drilled, sub-punched or sub-drilled and reamed, or thermally-cut (machine-guided). In the case of imperial bolts and fabrication practices, the hole is made 1.6mm (1/16in.) larger in diameter than the nominal size of the bolt, so the connection can slip slightly. Clause 22.3.5.1 of S16 allows 3/4-inch (19mm) bolts to be used in holes up to 22mm in diameter without penalty, so connection design should assume a 22mm diameter hole for a 19mm bolt. Similar allowances are made for other imperial bolt sizes. *Bearing-type connections* rely on the bolt acting as a pin to prevent large movements between the connected elements, while *slip-critical connections* rely on the bolts to clamp the connected parts together, creating a frictional force between the parts such that slip is prevented at service loads.

10.5 BEARING CONNECTIONS

The most common type of bolt for connection of steel parts in buildings is the 3/4-inch (19mm) diameter A325 bolt used in a bearing-type connection. For bearing-type connections, the shear and tensile capacity of these bolts can be taken from Table 3-4 of the *CISC Handbook*, reproduced for 3/4-inch (19mm) bolts in Table 10.2 (shear capacities are for single shear).

Table 10.2. Shear and tensile capacity of bolt

Bolt Size	Shear capacity threads intercepted	Shear capacity threads excluded	Tension Capacity
¾-inch Diameter	79.0 kN	113 kN	141 kN

Source: Table 3-4 of the *CISC Handbook*

The region of the bolt that does not have threads, called its shank, provides a greater resistance to shear than the threaded portion. S16 recognizes this and reduces the bearing resistance capacity of the bolt if the shear plane is intercepted by the threads. To use the threads-excluded bolt values requires more control on the part of the fabricator, but may result in fewer bolts being used. If connections are detailed on the design drawings using threads excluded, the drawings should clearly state that this is required.

Bolts can act in single or double shear, as shown in Figure 10.4. Generally, double shear connections require more fabrication, but as the bolt intersects two shear planes in double shear, each bolt can carry more load.

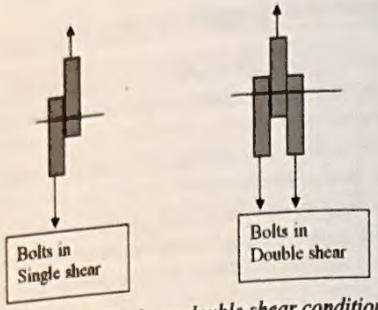


Figure 10.4. Single vs. double shear conditions

The capacity of the bolt is reduced if the connected parts are too thin due to bearing deformations that take place in the base material behind the bolt. For 3/4-inch (19mm) bolts acting in single shear (threads excluded), the capacity will be at its maximum if the connected steel is 350W or 300W (F_u = 450MPa) and is at least 6mm thick. For thickness of material less than 7mm, refer to Table 3-6 in the *CISC Handbook*. Bolts in double shear will require a material thickness of 12mm if the bearing stress is not to govern bolt capacity.

Connections designed for shear will typically also have to contend with an eccentricity. Under these circumstances the resulting direction on each bolt in a group will change as shown in Figure 10.5.

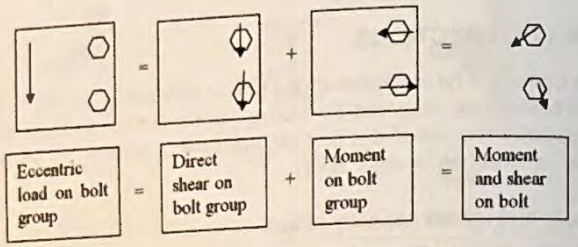


Figure 10.5. Change in shear direction in bolt groups

The force on bolt groups, especially those with several rows of bolts, can be complex and tables in the *CISC Handbook* assist with this task.

Minimum edge distances for bolts is given in Clause 22.3 and Table 6 of S16. For 3/4-inch (19mm) bolts, the minimum edge distance is 25mm at cuts that are rolled, sawn, or gas, plasma, laser, or water-jet and 32mm at sheared edges. To account for this requirement conservatively, the structural designer should consider sheared connecting plates and use 32mm. End distance (in the direction of the principal applied force) for tension members may be greater.

The minimum centre-to-centre distance between bolts (pitch) is 2.7 times the bolt diameter; however, 3.0 times the bolt diameter is preferred to facilitate assembly. Some fabricators standardize their details with a 75mm or 80mm pitch.

Bolted connections always use a minimum of two bolts to assist in erection and provide redundancy. The beam or connected element is lifted into place and held in position with the pointy end of a spud wrench or a drift pin in one bolt hole, while a bolt is placed in the second hole.

Examples of bolted connections where eccentricity of the load must be considered are shown in Figure 10.6.

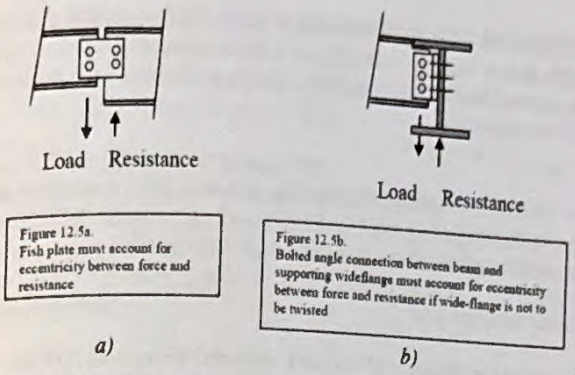


Figure 10.6 a) Fish plate must account for eccentricity between force and resistance. b) Bolted angle connection between beam and support wideflange must account for eccentricity between force and resistance if wideflange is not twisted

EXAMPLE—DETERMINE THE NUMBER OF BOLTS

Determine the number of bolts in a single vertical line that are required to support a 250kN shear acting at 200mm from the centre of the bolt line. Use 19mm (3/4-inch) A325 bolts with threads excluded in a bearing-type connection.

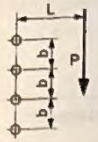
Solution:

The capacity of the 19mm (3/4-inch) diameter bolts in a bearing type connection with threads excluded is 113kN/bolt. Use Table 3-14 of the *CISC Handbook* for a single row of bolts and assume an 80mm pitch (see Figure 12.6). With a moment arm, L, of 200mm

$$C = P/V = 250kN / 113kN = 2.21$$

Using the bolt pitch of 80mm section, read down the table under L = 200mm until we find a value of C that exceeds 2.21 this occurs with five bolts with C = 2.19 (close enough). The connection requires five 19mm (3/4-inch) A325 bolts at a pitch of 80mm. See Table 10.3 (*CISC Handbook* Table 3-14).

Table 10.3. Eccentric loads on bolt groups



$$C = \frac{P_f}{V_r}, \text{ or } \frac{P}{V_s}$$

ECCENTRIC LOADS ON BOLT GROUPS
Coefficients C

Table 3-14

Moment Arm, L, mm												Number of Bolts	Pitch b mm
75	100	125	150	175	200	225	250	300	400	500	600		
0.94	0.74	0.61	0.51	0.44	0.39	0.35	0.32	0.26	0.20	0.16	0.13	2	
1.87	1.50	1.24	1.05	0.90	0.79	0.71	0.64	0.53	0.40	0.32	0.27	3	
2.96	2.51	2.14	1.86	1.63	1.45	1.30	1.18	0.99	0.75	0.61	0.51	4	
4.06	3.58	3.14	2.76	2.45	2.19	1.98	1.80	1.52	1.15	0.92	0.77	5	
5.16	4.68	4.20	3.76	3.38	3.06	2.78	2.54	2.16	1.66	1.34	1.12	6	80
6.25	5.78	5.29	4.82	4.39	4.00	3.66	3.37	2.89	2.23	1.80	1.51	7	
7.32	6.88	6.40	5.91	5.44	5.01	4.62	4.28	3.70	2.88	2.34	1.97	8	
8.38	7.97	7.50	7.01	6.53	6.06	5.64	5.24	4.57	3.59	2.93	2.47	9	
9.43	9.04	8.60	8.11	7.62	7.14	6.68	6.26	5.50	4.37	3.59	3.03	10	
10.5	10.1	9.68	9.22	8.73	8.24	7.76	7.30	6.48	5.20	4.30	3.64	11	
11.5	11.2	10.8	10.3	9.83	9.34	8.85	8.38	7.50	6.08	5.05	4.30	12	

EXAMPLE—DETERMINE THE NUMBER OF BOLTS TO SUPPORT SHEAR

Determine the number of bolts in a single vertical line that are required to support a 250kN shear acting at 200mm from the centre of the bolt line. Use 19mm (3/4-inch) diameter A325 Bolts with threads intercepted in a bearing type connection.

Solution:

The capacity of the 3/4-inch (19mm) bolts in a bearing type connection with threads intercepted by the shear plane is 79.0kN/bolt. Use Table 3-14 of the *CISC Handbook* (shown in Table 10.3) for a single row of bolts and assume an 80mm pitch. With a moment arm, L, of 200mm

$$C = P/V = 250kN / 79.0kN = 3.16$$

Read down the table under L = 200mm until we find a value of C that exceeds 3.16 this occurs with seven bolts. (The value of 3.06 for six bolts is too low.) The connection requires seven 19mm (3/4-inch) diameter A325 bolts at a pitch of 80mm if the shear plane intercepts the bolt thread.

10.6 SLIP-CRITICAL CONNECTIONS

Slip-critical connections require three things:

1) A special preparation of the contact surface between the connected elements referred to as the faying surface. (Painted or galvanized surfaces are not acceptable though under some limited circumstances some primers may be acceptable and galvanizing is acceptable in type C surfaces; see Clause 23.3.)
2) The use of A325 or A490 bolts. (A307 bolts are not acceptable.)
3) A tensioning of the bolts to provide a clamping force. The bolts must be tensioned to 70% of their specified minimum tensile strength of the bolt. This tightening requires greater inspection, including torque testing of the bolts.

If any of these three items is missing, then the connection will not be recognized as a slip-critical connection by S16.

The capacity of slip-critical bolts depends on the surface condition of the clamped parts. These requirements are given in Clause 23.3. The most basic is "clean mill scale" in which the surface must be free of oil, paint, lacquer, or any other coating in the bolt area and slightly beyond. Clean mill scale finish is classified as a Class A finish. A Class B finish requires blast cleaning.

For slip-critical connections, the bolt capacity is given in Table 3-11 of the *CISC Handbook* but, for the most common bolt – the 3/4-inch A325 bolt. The values are given in Table 10.4.

Table 10.4. Capacity for 3/4-inch A325 bolts

Bolt Size	Service load Shear capacity Class A surfaces	Service load Shear capacity Class B surfaces
Surface Condition	Clean Mill Scale	Blast Clean
19mm (3/4-inch) diameter A325 Bolts (regular hole or short slot)	33.7kN	56.1kN
19mm (3/4-inch) diameter A325 Bolts in long slot	25.3kN	42.1kN

Slip-critical connections are designed on the basis of making certain that the assembly does not slip at service loads and then checking the factored load conditions using the bolts as bearing bolts. Note that values in Table 10.4 show service load shear capacity at the top of the table.

Slot dimensions can be found in the *CISC Handbook* in Part 6 for metric-sized bolts. The width of the hole is the bolt dimension plus 2mm while the length is 4mm longer than this for a slot on a 3/4-inch (19mm) bolt and 28mm longer for a long slot on a 3/4-inch (19mm) bolt. A 3/4-inch (19mm) bolt may be in a hole size of 21mm or 22mm (see Clause 22.3.5.1).

EXAMPLE—DETERMINE THE NUMBER OF BOLTS IN SLIP-CRITICAL CONNECTION

Determine the number of bolts in a single vertical line that are required to support a 250kN shear acting at 200mm from the centre of the bolt line. The dead load is equal to the live load and the service load is 185kN. Use 3/4-inch A325 Bolts in a slip-critical connection with short slot holes. Consider the bolted surface to be clean mill scale.

Solution:

The capacity of the 3/4-inch (19mm) bolts in a slip-critical connection with clean mill scale finish is 33.7kN/bolt. Use Table 3-14 of the *CISC Handbook* for a single row of bolts and assume an 80mm pitch. With a moment arm, L, of 200mm

$$C = P/V = 185kN / 33.7kN = 5.50.$$

Reading down the table under L = 200mm until we find a value of C that exceeds 5.50. This occurs with nine bolts; thus the connection requires nine 19mm (3/4-inch) diameter A325 bolts at a pitch of 80mm. Each of these bolts would then be tightened to70% of their factored tensile capacity. The increase from the six bolts found for a bearing type connection to eleven bolts in a slip-critical connection shows why slip-critical connections are used only when there is no alternative.

Slip-critical connections are required in the following conditions:

1) When slotted holes are used and there is a shear to be transferred through an eccentricity.
2) Where there is fatigue loading or frequent load reversals.
3) Where the structure is extremely sensitive to deflections such that slippage into bearing cannot be tolerated.

10.7 BLOCK TEAROUT OF CONNECTIONS (TENSION AND BLOCK SHEAR FAILURE)

Block tearout of connections occurs where a shear failure plane passes through the centres of the bolt holes and provides a weak link in the connection resistance chain.
Figure 10.7 shows the type of failure that we are guarding against.

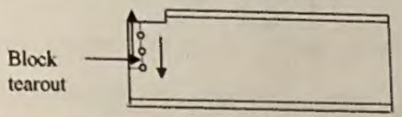


Figure 10.7. Intended behaviour of block tearout

New formulas have been included in S16-09 to cover this situation; Clause 13.11 covers it for gusset plates and for coped beams. These formulas are advancement from previous editions of S16 as they account for the effects of tension in the connection as well the net and gross length of the connection. The new formulas reflect research from the University of Alberta which shows that rupture on the tension face occurs before rupture on the shear face.

For the coped beam above, the formula of Clause 13.11 are follows:

$$T_r = \phi_u \left[U_t A_n F_u + 0.6 A_{gv} \frac{(F_y + F_u)}{2} \right]$$

Where: U_t = Efficiency factor for the conditions of interest we get:
$U_t = 0.6$ for angles connected by one leg to one piece and by the other leg to a second piece,
$U_t = 0.9$ for coped beams with single vertical row of bolts
$U_t = 0.3$ for coped beams for two bolt lines
$U_t = 0.0$ for coped beams with more than two bolt lines

These values, taken from the S16 commentary, are as shown in Figure 10.8.

No.	Connections subject to block shear	U_t
1	Coped beam with one row of bolts	0.9
2	Coped beam with two rows of bolts	0.3
3	Angle in tension connected by one leg	0.6
4	Angle in shear, one leg bolted to supported beam and other leg welded on 3 sides to supporting member	0.6
5	Single plate (shear tab) bolted to supported beam, welded to supporting member	0.6

Figure 10.8. Connections subject to block shear
Source: S16 Commentary

A_n = Net area in tension
A_{gv} = Gross area in shear
$\emptyset_u = 0.75$

When using these formulas it is necessary to decide which parts of the connection are in shear and which parts are in tension. See Figure 10.9, which is taken from the *CISC Handbook* and modified.

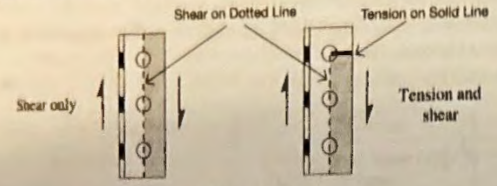


Figure 10.9. Tension and block shear failure

Tension and Shear Block Failure
S16 Clause 13.11

As shown in Figure 10.9 there may be more than one failure plane and when evaluating block failure it is necessary to choose the failure path with the least capacity.

Refer also to Clause 13.11 for block failure in gusset plates, a similar modification where no tension exists.

EXAMPLE—DESIGN A BOLTED CONNECTION

Design a bolted connection for a W460x74 350W steel beam spanning 7.5m. Assume an eccentricity of 75mm between the centreline of the bolts and the centreline of the supporting beam. The top flange of the beam is coped out 35mm.

Step 1: Determine the design load for the connection. Use beam load tables in Part 5. The capacity of a laterally supported W460x74 beam with a 7.5m span is 546kN. The connections will therefore be designed for 546kN / 2 = 273kN each end.

Step 2: Select the bolts. 3/4-inch A325 bearing type bolts will be used with threads excluded from the shear plane. Shear capacity of bolts is given in Table 3-4 of the *CISC Handbook*. A325 bolts have a shear capacity of 113kN provided the threads are excluded from the shear plane.

Step 3: Check the bearing resistance of the bolts in the beam web. W460x74 has a web thickness of 9.0mm. Bearing resistance for 3/4-inch bolts is 155kN from Table 3-6 of the *CISC Handbook*. (Bearing capacity of the web is OK for single shear but will govern with double shear.)

Step 4: Select the connection type. Use a single angle connection that has a single shear plane. Eccentricity between the bolts and the end of the angle is 75mm.

Step 5: Determine the number of bolts for eccentric loads on a single line of bolts on Table 3-29 of the *CISC Handbook*. For V_f = 113kN, P_f = 273kN, $C = P_f / V_f = 273 / 113 = 2.42$. Need 4 bolts providing a C = 2.95. Length of angle = 80mm *(4 – 1) + 40mm + 40mm = 320mm. (OK fits in the 395mm available in the distance between fillet welds of the W460x67 (Dimension T in the W shapes beam properties table in Part 6).

Step 6: Check block tearout at the coped end A_{gv} is the gross area in shear and taken as the beam shear area less the coped portion.

The depth of cope is 35mm and the width of the web on the W460x67 is 9.0mm. Bolt holes for 19mm bolts taken as 22mm and centre of bolts from end of beam taken as 35mm.

$A_{gv} = d*w - w*cope$

$A_{gv} = 457mm*9.0mm - 9.0mm*35mm$

$A_{gv} = 4113mm^2 - 315mm^2$

$A_{gv} = 3798mm^2$

A_n = Net area in tension = 35mm*9mm = 315mm²

U_t = Efficiency factor = 0.9 for coped beam with single vertical row of bolts

$$T_r = \emptyset_u \left[U_t A_n F_u + 0.6 A_{gv} \frac{(F_y + F_u)}{2} \right]$$

Placing the values of A_n and A_g developed above in the S16 equation for block tearout we get:

$$T_r = 0.75 \left[0.9*315mm^2*10^{(-6)}*450{,}000kN/m^2 + 0.6*3798mm^2*10^{(-6)}*\frac{350{,}000+450{,}000}{2} \right]$$

$T_r = 0.75(1039kN) = 779kN$

(OK>273kN)

Away from bolt holes: $V_r = 0.60 \emptyset A_{gv} F_y$

$V_r = 0.60 * 0.9 * (4113 - 315) \times 10^{-6} m^2 * 350{,}000 kN/m^2$

$V_r = 717kN$ (OK > 273kN)

Step 7: Minimum size of angle – check bearing and shear.

Need 6mm for bearing resistance (See appropriate table in *CISC Handbook*.) (Bolt load = 273kN/2.95 = 92.5kN)

Shear resistance of the angle from S16 Clause 13.11

A_{gv} = 320mm *6.4mm = 2048mm^2

A_n = 0mm^2 Tension line similar to Figure 12.8 left-hand side.

$$T_r = \phi_u \left[U_t A_n F_u + 0.6 A_{gv} \frac{(F_y + F_u)}{2} \right]$$

Simplify as A_n = 0.0

$$T_r = \phi_u \left[0.6 A_{gv} \frac{(F_y + F_u)}{2} \right]$$

Substitute A_{gv}, F_y, and F_u

$$T_r = 0.75 \left[0.6 * 2048 mm^2 * 10^{-6} * \frac{350{,}000 + 450{,}000}{2} \right]$$

T_r = 0.75[491kN] = 369kN (OK > 273kN)

Use 6.4mm (1/4-inch) angle.

Step 8: Call up the connection: four 19mm (3/4-inch) diameter A325 bolts – threads excluded at 80mm pitch. Single angle connection use L127x127x6.4.

10.8 CONNECTION OF BRACES NOT DESIGNED FOR YIELDING

When we reviewed tension capacity of members in Chapter 9 we looked at the case of a L76x51x6.4 double angle brace and found that the gross section capacity of the brace was 414kN while the net section capacity was 422kN. If we were to conclude this example and look at the connection at the end of the brace which involves three 19mm bolts in a row through one of the legs of each angle we would follow equations ii and iii of Clause 13.2.

Block Shear

$$A_n = \frac{76mm - 22mm}{2} (6.4mm) = 173mm^2$$

$$A_{gv} = (80mm + 80mm + 40mm)(6.4mm) = 1280mm^2$$

U_t = 0.6 for angles connected

$$T_r = \phi_u \left[U_t A_n F_u + 0.6 * A_{gv} \frac{(F_y + F_u)}{2} \right]$$

For each of the two angles this reduces to:

$$T_r = 0.75 \;\; 0.6(173x10^{-6} m^2) 450{,}000 \frac{kN}{m^2} + 0.6 * 1280x10^{-6} m^2 \frac{(300{,}000 + 450{,}000)}{2} \frac{kN}{m^2}$$

T_r = 0.75[46.7kN + 288kN] = 251kN

For two angles T_r = 2 * 251kN = 502kN

Net section

$T_r = \phi_u A_{ne} F_u$

$A_{ne} = 0.6 A_n$ for angles connect by only one leg with three bolts in one row (Clause 12.3.3.2 (b)(ii))

T_r = (0.75)(0.6)(625x10-6m^2)(450,000kN/m^2)

T_r = 127kN

For two angles T_r = 2*127kN = 254kN

The design summary is shown in Table 10.5.

Table 10.5. Design summary for 2-L75x51x6.4 braces

Condition	Resistance
• Gross Section	T_r = 414 kN
• Net section at intermediate bolts	T_r = 422 kN
• Shear block and end connection	T_r = 502 kN
• Net section and end connection	T_r = 254 kN

It is worth noting that the net section at the end connection for the member is significantly lower than all the other values and often govern due to shear lag effects.

10.9 THE EFFECT OF PRYING ACTION ON BOLT LOADS

Under some circumstances, tension on an element can pry against the bolts connecting that item and increase the loads on those bolts. Bolts in tension and prying action are covered in the connection section of the *CISC Handbook* along with some worked examples, as shown in Figure 10.10.

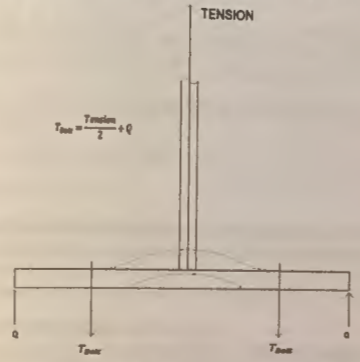


Figure 10.10. Bolts in tension and prying action

The prying effect is worse if the plate that is bolted is flexible as the value of Q will increase and therefore the tension on the bolt. Use the formulas and charts in the *CISC Handbook*. The desire is not to overload

the bolts in tension. The *CISC Handbook* has an example where the bolt tension is increased by 35% by prying action.

10.10 A307 ANCHOR RODS AND BASE PLATE CONNECTION TO CONCRETE

A307 bolts are frequently used for anchor rods when connecting steel elements to concrete. Anchor rods have a ϕ_w of 0.67, so if we look up the capacity of the bolt in Table 3-4 of the *CISC Handbook*, we find incorrect values because Table 3-4 uses ϕ_b of 0.8.

Table 10.6 calculates the shear and tension resistance of the anchor rods in accordance with the requirements of S16-09 Clause 25.

Use caution when evaluating the formula for A_n (the net tensile area of the rod) for imperial rods as the formula is in imperial units, d must be in inches, and the area produced is in square inches.

Table 10.6. Anchor rod capacities (kN per bolt) (S16)

A307 Anchor Rods								
Phi = 0.67		Follow the requirements of Clause 25						
Fu = 414		n = threads per inch from table in Part 6						
Fy = 300								
Diameter	Diameter	n (threads /inch)	A_n	A_n	T_r	A_{ar}	V_r	V_r
Inches	Decimal inches		(in^2)	(mm^2)	(kN)	(mm^2)	Thread excluded	Thread included
5/8-inch	0.625	11	0.226	146	40.4	198	32.9	23.1
3/4-inch	0.750	10	0.334	216	59.9	285	47.4	33.2
7/8-inch	0.875	9	0.462	298	82.6	388	64.6	45.2
1-inch	1.000	8	0.606	391	108.4	507	84.3	59.0
1-1/8-inch	1.125	7	0.763	492	136.6	641	106.7	74.7
1-1/4-inch	1.250	7	0.969	625	173.4	792	131.8	92.2
1-1/2-inch	1.500	6	1.405	907	251.5	1140	189.7	132.8

The formulas for combined shear and tension on an anchor rod are given in S16-09 Clause 25.3.4 as follows:

$$\left(\frac{V}{V_r}\right)^2 + \left(\frac{T_f}{T_r}\right)^2 \le 1$$

EXAMPLE—DESIGN A BASE PLATE CONNECTION TO CONCRETE

A column at the bottom of a lightly loaded brace bay has a factored uplift of 90kN and a factored shear of 100kN. It is proposed to use a base plate with four 3/4-inch A307 anchor rods. Determine if this meets the requirements of Clause 25. The base plate is stiff enough that we can ignore prying.

Solution:

Let's conservatively assume the anchor rod threads intersect one of the shear planes. For each bolt T_r = 59.9kN and V_r = 33.2kN plug these values into the equation above.

$$\left(\frac{100kN}{4*33.2kN}\right)^2 + \left(\frac{90kN}{4*59.9kN}\right)^2 = (0.75)^2 + (0.38)^2 = 0.71 \text{ (OK <1)}$$

As there is no way that a 3/4-inch anchor rod has the bending capacity to take the shear from the base plate to the point in the concrete where it can be resisted by bearing, it is also important to evaluate the capacity of the shear transfer using shear friction methods. Using the formulas of A23.3 Clause 11.6, and transforming from a stress formulation to a force formulation we get the following:

$$V_r = \phi_c \mu (A_b F_y + N)$$

Where ϕ_c = 0.65

μ = Coefficient of friction

N = Axial force acting on the assembly positive for compression

$(A_b F_y + N)$ is a clamping force

The values of μ for the shear friction values between steel base plates and grout can be obtained from A23.3 appendix D Clause D4.3.5 giving the following values:

μ = 0.8 if the contact plane is a full plate thickness below the top surface of the concrete. (This would be the situation for a base plate buried in concrete.)

μ = 0.6 if the contact plane is level with the top surface of the concrete.

μ = 0.5 if the contact plane is above the level of the surrounding concrete. (This is the usual situation where the base plate is above the level of a slab or on a pilaster. Note that his case was omitted from more recent editions of A23.3.)

EXAMPLE—DESIGN A BASE PLATE CONNECTION TO CONCRETE WITH WIND LIFT

Continue the previous example with a factored uplift of 90kN and a factored shear of 100kN and check shear friction with four 3/4-inch (19mm) A307 anchor rods. The base plate is above the surrounding slab and sits on a 25mm layer of steel shims and grout.

Solution:

$V_r = \phi_c \mu \left(A_b F_y + N\right) = 0.65 * 0.5 * (4 * 285 * 300 \text{ x } 10^{-3} - 90kN)$

$V_r = 0.325 * (4*85.5kN - 90kN)$

$V_r = 0.325\ (252kN) = 75.6kN$ (No Go this is less than the 100kN applied)

Try four 1-inch diameter A307 anchor rods

$V_r = 0.3 * (4*152.1kN - 90kN) = 167kN$ (OK >100kN

Use four 1-inch diameter anchor rods)

For small shears like 100kN we can probably get away with doing shear and tension transfer with anchor rods but at higher values it will be necessary to use shear lugs which provide a more direct method of transferring the shear force to the concrete.

10.11 BOLTED CONNECTIONS UNDER S16

Bolts have a ϕ_b of 0.8 this applies to metal to metal bolts but not to the bearing capacity of the bolt.

The term "anchor bolts" is no longer used and bolt-like objects connecting base plates to concrete are called "anchor rods" and have ϕ_w of 0.67.

Formulas in Clause 13.11 cover shear and tension block failure and the formulas that covered shear only on connector plates has been removed.

10.12 BASIC WELDED CONNECTION DESIGN

The following are very basic guidelines for the design of steel welded connections for beams for building use in Canada. It is intended that this chapter be read in conjunction with the *CISC Handbook*, where additional information on welded connections is found.

Welds are used in many connections, especially those that can be welded in the shop where access, equipment and quality control are optimal.

The most common type of welded connection is the fillet weld in which a triangular bead of weld is added at the corner intersection between the two steel parts to be joined, as shown in Figure 10.11.



Figure 10.11. Triangular bead in fillet weld

There are two important dimensions in the fillet weld are as follows:

1) Dimension D – The weld size. This is referred to in the capacity tables
2) Dimension t_e – The throat size

The dimensions are shown in Figure 10.12.

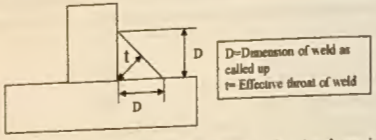


Figure 10.12. Fillet weld dimensions

Where a fillet weld is used against the edge of a plate (as shown in Figure 10.11b), the maximum size of fillet weld must be less than the plate thickness, as shown in Table 10.7.

Table 10.7. Maximum weld size at the edge of a plate

Plate edge condition	Maximum D
Sheared edge t <= 6mm	t
Sheared edge t > 6mm	t-2mm
Rolled edge	0.75t

Weld symbols can be found in Part 6 of the *CISC Handbook*, which gives a great deal more options than most designers are comfortable calling up. For example, for a fillet weld the basic symbols are shown in Figure 10.13.

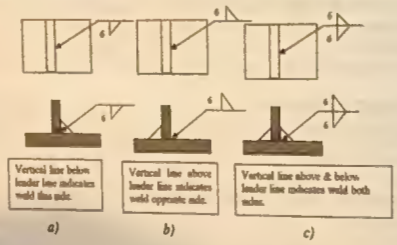


Figure 10.13. Basic weld symbols

Two other weld symbols of use are the weld-all-around symbol and the field weld symbol, as shown in Figure 10.14.

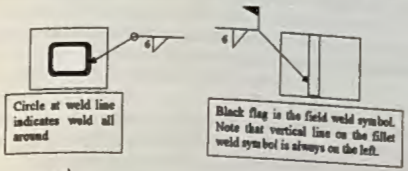


Figure 10.14. Additional weld symbols

The capacity of fillet welds is given in Table 3-24 of the *CISC Handbook* for both imperial- and metric-sized fillet welds. 300W and 350W material is usually welded with E49XX (this being a new and slightly improved version of E480XX) welding rods. The capacity for a 6mm weld (when the angle between the weld and force is 0 degrees) is 0.933kN/mm. Weld capacities for larger fillets can be determined by taking a ratio of the weld size to 6mm. It is therefore only necessary to remember one number to know the weld capacities of all fillet weld sizes (e.g., a 10mm fillet has a capacity of (10/6) * 0.933kN/mm = 1.55kN/mm). For a fillet weld between pieces that are connected at right angles, the size of the fillet weld is the vertical or the horizontal dimension.

The steel standard recognizes the increase in capacity of the weld depending on orientation. Forces on welds in connections are often complex, and there is a desire to be conservative and quick in connection design. Also, many designers were trained before these clauses were incorporated in the Code. Consequently, many designers will conservatively use the values for 0 degrees between the axis of the weld and the force, as shown in Figure 10.15.

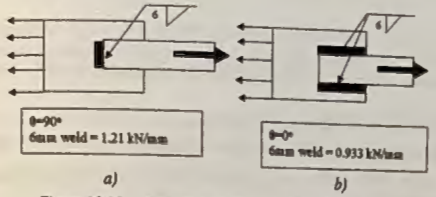


Figure 10.15. Capacity of weld depending on orientation

Regardless of the force on the weld, the weld size must satisfy the minimum weld size required for the thicker piece of steel being joined. This requirement is set out in Part 6 of the *CISC Handbook* and is as shown in Table 10.8. The principal reason for these requirements is that a very thick plate can draw the heat out of a small weld very quickly, creating the potential for weld cracking.

Table 10.8. Minimum weld sizes for various plate thicknesses

Material thickness of thicker part joined	Minimum size fillet weld (mm)
≤ 12mm	5mm
12mm < t ≤ 20mm	6mm
> 20mm	8mm

When applying Table 10.8, a thin section to a larger section the welding code W59 states that "the weld size need not exceed the thickness of the thinner part joined, unless a larger size is required by calculated stress."

Similar to the tables that assisted us with bolted connection design, the *CISC Handbook* contains tables that assist with the design of welded connections.

EXAMPLE—DESIGN A FILLET WELD

A 12mm thick fin plate from 300W material is welded onto a vertical steel surface that is perpendicular to the surface using a fillet weld each side of the plate as shown in Figure 10.16. Determine the welded connection necessary to support 250kN at 100mm from the vertical support face.

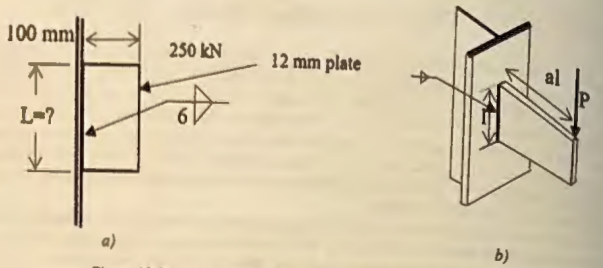


Figure 10.16. Fin plate welded onto a vertical steel surface that is perpendicular to the surface using a fillet weld each side of the plate

Solution:

Using Table 3-34 of the *CISC Handbook* we desire to find the weld length l and fillet size. The *CISC Handbook* example that accompanies Table 3-34 has the length of weld assigned and we must iterate to find the correct answer. Our factored load is P = 250kN and our eccentricity is a*l = 100mm.

Try l = 200mm a = (e = 100mm)/ (l = 200mm) = 0.5

From Table 3-34, using a 12mm plate and a 6mm weld and a = 0.5, we find C' = 1.20

l required = 250kN / 1.20 = 208mm (our guess was very close to the correct solution – they don't always come out this close).

Note that as the length of weld increases the ability of the weld to resist the eccentric moment improves. Some designers might argue that 200mm is close enough to the answer an live with a small overstress. Rather than using 208mm we will try 225mm.

Try l = 225mm a = (e = 100mm)/ (l = 225mm) = 0.44

From Table 3-34, using 6mm weld, C' = 1.28 l required = 250kN / 1.28 = 196mm (OK)

Result:

A 12mm plate with a 225mm length and a 6mm weld in which each side would have sufficient weld capacity.

Check stress in the plate: M = 250kN * 0.1m = 25.0kNm

Stress = 25.0kNm / ($bh^2/6$) = 25.0kNm / (0.012m * (.225m)^2 / 6) = 246,913kN/m^2 (OK)

Note that the section modulus ($bh^2/6$) as opposed to the plastic section modulus ($bh^2/4$) was used for checking stress as it was assumed that the plate was not yielding.

Note also that if the plate got too slender we may also want to consider lateral buckling. See Chapter 3 on unbraced beams for the buckling of an unsupported plate.

As the capacity of the plate should be at least the capacity of the sum of the welds, one other way of checking the plate thickness is to check that with a fillet weld D each side of a plate of thickness t, the thickness should be at least:

$$t \geq \frac{2*D}{1/Cos(45)} = \frac{2*D}{1.414}$$

In our case this is $t \geq \frac{2*D}{1.414} = \frac{2*6mm}{1.414} = 8.48mm$ (OK<12mm)

EXAMPLE—DESIGN A FILLET WELD ON THREE SIDES OF A PLATE

A 12mm thick fin plate from 300W material is welded onto a vertical steel surface that is parallel to the surface using a fillet weld on three sides of the plate. The plate is 300mm long and has 60mm horizontal welds at top and bottom as shown in Figure 10.17. Determine the welded size necessary to support 250kN at 100mm from the end of the horizontal fillet welds.

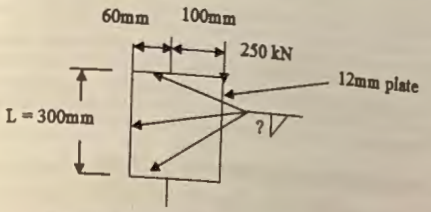


Figure 10.17. Fin plate welded onto a vertical steel surface

Solution:

Using Table 3-28 of the *CISC Handbook* we desire to find the fillet size. Our factored load P = 250kN. Using the geometry we find the eccentricity of the loads to the centre of mass of the welds.

$$a*300mm = 100mm + \frac{2*60*(60/2)+300*60}{(2*60)+300} = 151mm$$

a = 151mm / 300mm = 0.5k = 60mm / 300mm = 0.2

Using these two values and Table 3-28 of the *CISC Handbook*, we find C is 0.147

$$D = \frac{P}{C*l} = \frac{250kN}{0.147*300} = 5.7mm$$ use 6mm weld

10.13 CONNECTIONS USING A COMBINATION OF WELDS AND BOLTS

Sometimes it is desired to increase the capacity of a bolted connection by welding the connection plates to the beam. Because it is difficult to analyze welds in combination with bolts, Clause 21.10 allows welds to be used in combination with slip-critical bolts in new work, provided that the different ductilities of the bolts and the transverse and longitudinal welds are accounted for in accordance with Clause 13.14.

10.14 BUTT JOINTS AND DEVELOPING THE WALL OF A HSS TUBE

For handrails and other "non-structural" design, it is often necessary to try to develop the capacity of the wall of an HSS tube so the tube can carry moment. The HSS tube is frequently welded to an embed plate that is perpendicular to the HSS in a butt joint configuration as shown in Figure 10.18.

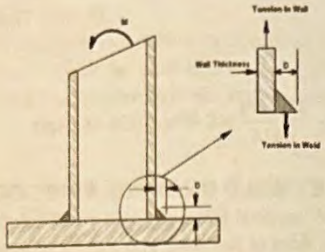


Figure 10.18 Welding a HSS tube

The tension in the wall is $\emptyset F_y t$, where t is the thickness of the wall of the HSS.

The capacity of the weld is that for a 90-degree weld (1.21kN/mm for a 6mm weld).

However, W59 (Clause 4.3.1.6.1.4) states that the maximum effective throat thickness of a partial penetration weld (such as a fillet weld) should not be greater than the wall thickness of the HSS member.

Using the values shown in Table 10.9, we see that the maximum capacity of a butt joint is only 91% of the wall strength. It may be necessary to increase the thickness of the wall of the HSS should the weld not be capable of developing the moment required to satisfy guardrail loads.

Table 10.9. Weld required for various thicknesses of HSS wall

Wall Thickness (mm)	**3.2**	**4.8**	**6.4**	**8.0**	**9.5**	**13.0**
Wall Tension (kN/mm)	1.01	1.51	2.02	2.52	2.99	4.10
Weld Required (mm)	4.96	7.44	9.91	12.39	14.72	20.14
Weld Used (mm)	**5.0**	**8.0**	**10.0**	**12.0**	**15.0**	**20.0**
Weld throat (mm)	3.54	5.66	7.07	8.48	10.61	14.14
Maximum effective throat (mm)	3.2	4.8	6.4	8.0	9.5	13.0
Weld Capacity (kN/mm)	0.92	1.38	1.84	2.30	2.73	3.74
Capacity Developed (mm)	91%	91%	91%	91%	91%	91%

In Table 10.9, we also see that the weld size gets very thick once the wall gets thick. We will look at the connection of HSS members for axial loads in Chapter 15 on designing bracing for seismic loads.

For normal guardrail posts, however, it may be possible to increase the capacity of the weld by realizing that the tension result for the weld is outside the wall of the post being connected and computing the moment result of the weld stress at that point.

EXAMPLE—DETERMINE THE MOMENT CAPACITY

Determine the moment capacity of a 8mm weld on a 48mm diameter x 4.8 HSS (G40.21-Grade 350W) post subject to moment and take into account the offset between the centre of the weld and the centre of the wall of the HSS as shown in Figure 10.19.

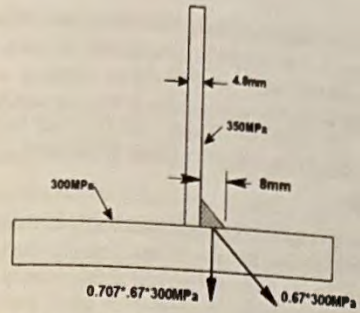


Figure 10.19. Offset between the centre of the weld and the centre of the wall of the HSS

$Z = 9.0x10^3$ mm^3 $M_r = 2.86$kNm

The diameter of the weld circle is 48mm on the inside and 48 + 8 + 8 = 64mm

The plastic section modulus of a hollow circle is $Z = (1/6)(d_o^3 - d_i^3)$

$Z = (1/6)(64^3 - 48^3) = 25.2 \times 10^3$mm^3

Assuming the base plate is from 300MPa material and taking the component of vertical load and applying $\emptyset_w = 0.67$ we get a stress of 300,000kN/m^2 * 0.707 * 0.67 = 142,100kN/m^2

Applying this stress to the Z of the weld material we get a moment resistance M_r = 142,100kN/m^2 * $25.2x10^3$ mm^3 = 3.58kNm

Limit to the plastic capacity of post = 2.86kNm

Repeating the above calculation using a 6mm weld results in a moment capacity of only:

M_r = 142,100kN/m^2 * (1/6)($60^3 - 48^3$) = 2.50kNm (<M_r of 2.86kNm)

Final Result: Use 8mm weld to develop the moment capacity of the tube while satisfying the maximum weld throat for the 4.8mm wall.

10.15 CALLING UP PLATES

Note: Thanks for input from three very knowledgeable people who have spent many years in the steel fabrication business: Tim Fraser, Thor Gaul, and Rob Third.

Before 1980 the *CISC Handbook* recommended that plates be called up as Width x Thickness x Length. In 1980 the recommended wording for calling up plates on drawings changed to Thickness x Width x Length.

There are significant savings to make if the plate can be cut from bar stock where it needs only a single cut to be cut to length. Plate sections that must be cut on two sides are more labour intensive and should be avoided if a suitable bar stock piece can be selected. Bar stock is typically rolled in imperial measurements for both width and thickness. Two common plate thicknesses called up on drawings are 10mm and 12mm. The 10mm plate will often be substituted with 3/8-inch (9.5mm) while the fabricator will, if possible, substitute 1/2-inch (12.7mm) bar where a 12mm plate has been called up on a drawing. If the structural drawings have intended to use 1/2-inch plate but have rounded to 13mm, then the fabricator may send a Request for Information (RFI) asking if 12.7mm plate can be substituted or if more expensive 9/16-inch (14.3mm) plate has to be used. Preference should be given to selecting plates or bars that are the metric equivalent of 1/8-inch, 1/4-inch, 3/8-inch, 1/2-inch, 5/8-inch, 3/4-inch, or 1-inch or in half-inch increments thereafter.

Widths of bars come in half-inch increments to 6 inches wide and in 1-inch increments thereafter to a maximum of 12 inches. See Chapter 1 for a table of the available bar sizes.

Two steel fabricators indicated that, while plate should be called up with the thickness first, it is appropriate to call up bars with the width first as this is easier from the standpoint of material takeoffs.

10.16 GUSSET PLATES

Gusset plates are sometimes used to connect elements, particularly in brace bays where the plates can become quite large. When in the brace bay, the gusset plate is part of the seismic resisting system and should be detailed by the design engineer; however, in practice, the sizing of the gusset plate is often left to the detailer.

Check the following:

1) Shear lag to verify that there is enough connection distance between the brace element and the gusset plate.
2) Tension failure of the gusset plate (away from the bolt holes if bolt holes exist).
3) Tension failure of the gusset plate in bolt hole area if these occur.
4) Buckling failure of the gusset plate if the brace can go into compression.
5) For seismic braces, determine that the end distance is sufficient to provide ductile rotational behaviour in accordance with the requirements of Clause 27.5.4.3. (This will be covered more in Chapter 15.)

A section on connecting a plate at the end of brace is included in Part 3 the *CISC Handbook*. This section refers to checks for shear lag, yielding of the plate, and failure in the plate across the bolt line.

The *CISC Handbook* example is only partly appropriate to the design or checking of a gusset plate, as typical gusset plates usually taper and get smaller toward the brace, and typical brace members have compression forces that must be considered in the gusset plate.

Additional information on gusset plates and cyclic testing can be found in Cheng, Grondin, and Yam's "Design and Behavior of Gusset Plate Connections," a paper presented at the AISC Fourth International Workshop on Connections in Steel Structures, held October 22 to 25, 2000, in Roanoke, Virginia. The paper is available in the Other Technical Resources section of the ePubs and Freepubs page on the AISC website.

The AISC reference shows that the effective width of the gusset plate can be considered as the "Whitmore Effective Width," which is shown in Figure 10.20 as the line that is 30 degrees from the first bolt line. The AISC reference shows that the 30 degrees could be considered to be 45 degrees.

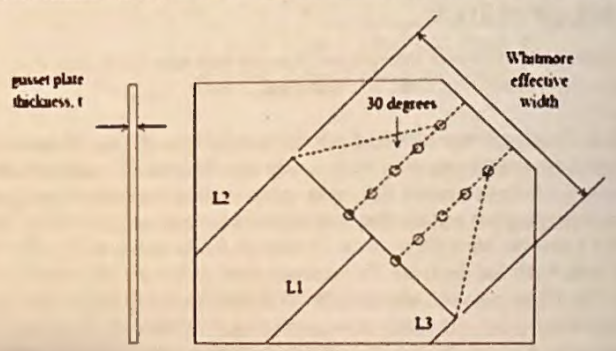


Figure 10.20. Whitmore effective width

Source: Cheng, Grondin, and Yam. "Design and Behavior of Gusset Plate Connections" (2000)

This reference shows the consideration of the various failure methods on the gusset and effects of cyclic loading but does not have a numerical example.

10.17 CONNECTIONS OF STRUCTURAL STEEL TO CONCRETE ELEMENTS AND OTHER MATERIALS

Almost all steel structures are connected to concrete at some point. If the building is completely constructed from steel, this connection will be at the foundation level using anchor rods and base plates. However, if many of the vertical load carrying elements of the building are constructed from concrete, then it may be necessary to transfer the load of beams to the face of a concrete wall or to make other attachments to concrete elements. These connections must be fully detailed by the design engineer on the structural drawings. As the contractor will be reluctant to have formwork damaged from projecting bolts, it is desirable to design the connection such that any elements that project from the wall are added after the formwork has been stripped. There are two products to achieve this aim:

1) An embedded steel plate with shear studs (Nelson studs) welded to the rear of the plate. This is embedded in the concrete at the time of concrete placement and attachment of the steel is made by welding to the embedded plate.
2) By the use of drilled expansion anchors (Hilti™ Kwik Bolts or Hilti™ HSL bolts being the most common).

While the connection to the embedded plate may be designed by the steel detailer, any embedded items must be designed and detailed by the engineer of record. The design of these items usually involves an interaction between information provided by the stud or insert manufacturer, the steel standard, and the concrete code. Examples of design using these products can be found in the product literature. An example of a detail for connection of a steel beam to an embedded steel plate is shown in Figure 10.21. The roof diaphragm chord is attached to the wall with an expansion anchor.

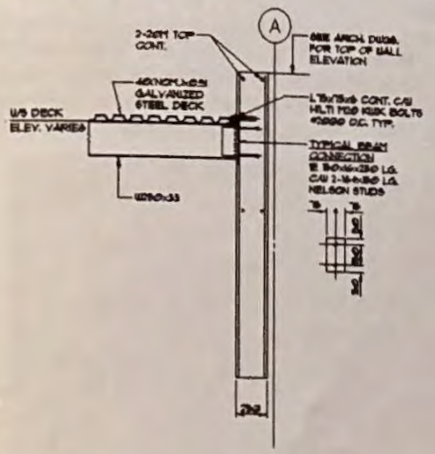


Figure 10.21. Detail for connection of a steel beam to embedded steel plate

In the same way that the structural drawings must fully detail the connections between steel and concrete, the drawings must show connections between any other materials, including between steel and wood, between steel and concrete, and between steel and masonry. The detailing is required for both steel supporting wood elements (e.g., glued-laminated [Glulam] beams picked up by a steel beam or column) and for wood elements supporting structural steel (e.g., a Glulam column picking up a steel beam).

CHAPTER 11
MISCELLANEOUS METAL

We have covered the design of the main vertical system for the building and shown how to design the deck, the beams and joists, and the columns. However, no building design would be complete without consideration of the "miscellaneous metal." These include canopies, stairs, guardrails, and a selection of items that are expected to show up on the structural drawings but do not appear to be part of the "main structure" of beams and columns. Often the designer can spend as much time designing and detailing the miscellaneous steel as on designing the main bones of the building. The level of detailing of these items will have a significant effect on how your client (usually the architect) views your work, and the successful design of these items can often mean the difference between getting or not getting the next job with that client. Miscellaneous metal can also be a significant source of income for the fabricator, who will claim extra fees if these items are not shown on the contract documents or are poorly coordinated with the architectural drawings.

11.1 STAIR DESIGN

Stair design is a shared design between the architect and the structural engineer. The architect will usually lay out the stair geometry giving the rise and run and a full dimensional layout, while the structural engineer will size and show the stringers and other members supporting the stair as well as the construction of treads and landings. Much of the stair design is done with typical details such as those shown in Figure 11.1.

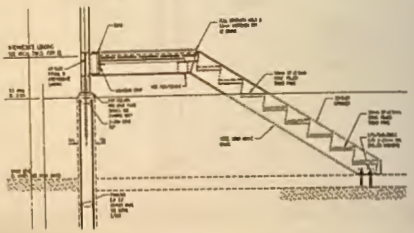


Figure 11.1 Typical framing for a short steel stair supported at grade on the low side and on steel structure on the high side. Note that the stair stringer is supported with a full-strength splice at the crank

Many stairs such as the one shown in Figure 11.1 use premanufactured bent-plate stair pans for treads and risers. These are shop-welded to the channel stringers that form the main beams of the stair, and each run of the stair arrives on site as a pre-fabricated steel unit. The steel erector puts the steel part of the stair in place and the general contractor adds the concrete to the treads and landings. The channel size is chosen to give sufficient depth to get the pans in place without having them protrude beyond the edge of the channel. At least some tolerance is provided in the channel measurement to make sure the treads fit into place. The main stringers for the stair are made from channel sections, which allow a straight face for attaching the stairs and eliminate the small pockets that would be present if the stringers were constructed from wide-flange sections. Figure 11.2 shows a stair constructed with a wide-flange stringer beside one constructed from a channel stringer.



a) *b)*

Figure 11.2. a) Typical framing for steel stairs uses channel sections for the stringer as this gives a clean edge to the stair. b) The wide-flange stringer shown at the right is much more difficult to detail and fabricate and creates pockets where dirt will collect

Most steel stairs located in public areas will have bent-plate stair pans (with the treads filled with concrete after the steel structure is constructed). In industrial areas, stairs are made from off-the-shelf treads as shown in Figure 11.2a, where steel pans are filled with concrete after the stair is erected, or sometimes from metal gratings. Typically, landings on stairs with concrete treads will be from concrete on metal deck.

Figure 11.3 shows a small stair in an industrial application.



Figure 11.3. Small stair in an industrial application

Many of the requirements that apply to stairs that are accessible by the public do not apply in industrial applications. Nevertheless, stairs would still be designed for a live load of 4.8kPa.

EXAMPLE—CHECK STAIR STRINGER

A 1500mm wide stair shown in profile in Figure 11.4 (rise of 2500mm and run of 3900mm) is to be constructed with concrete-filled metal pans and channel stringers on each side. On a plan view basis, the live load is 4.8kPa and the dead load is 2.0kPa, including stringer weight and concrete on metal pan. Check that the C310x31 stringer meets strength and deflection requirements.

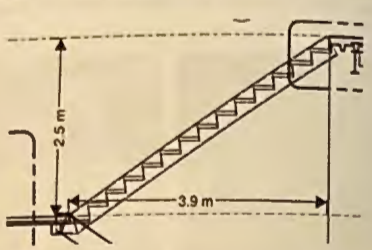


Figure 11.4. Concrete-filled metal pan stair with channel stringers

Solution:

The first step is to realize that the load is vertically downward while the stringers are inclined. This means that only a portion of the load is perpendicular to the stringer. The portion of the load that is parallel to the stringer will give rise to generally small axial forces and a need to anchor at the base of the stair but; for an evaluation of the stringer, we are mostly interested in the bending that is caused by the portion of the load perpendicular to the stringer.

The slope of the stringer is ArcTan (Rise/Run) = ArcTan (2.5m / 3.9m) = 32.7 degrees

The length of the stringer from geometry is 4.63m (square root of $(2.5m)^2 + (3.9m)^2$)

The vertical load = (1.5m/2) * (1.25 * 2.0kPa + 1.5 * 4.8kPa) = 7.275kN/m

The load perpendicular to the stringer is 7.275kN/m * Cos(32.7 deg.) = 6.12kN/m

Factored moment = $WL^2 / 8$ = 6.1kN/m * $(4.63m)^2 / 8$ = 16.34kNm

We consider the pans that are welded to the stringer provide lateral restraint and the channel is considered to be laterally supported.

Moment resistance of a C310x31: $M_r = \phi F_y S_x$ = 0.9 * 350,000 * 351 x10^{-6} = 110kNm

Strength is about six times the load imposed, but we will continue to use this section as a smaller size section would not provide sufficient end closure for the stairs.

Check deflection:

Deflection is not an issue.

The C310x31 works. While it would be possible to find a smaller section that would work for strength and deflection, the C310x31 is used because it provides sufficient depth for the pans to be fully covered and is the lightest weight C310 that is available. While lighter weight "Miscellaneous Channels" such as MC310x15.8 do exist, the C310x31 is used because it is easier to source.

11.2 HANDRAILS AND GUARDRAILS

Handrails and guardrails are often a challenge for the structural engineer as these are often very visual items that are often an architectural feature. However, they must satisfy several Building Code–specified configuration requirements and be designed for significant forces. The configuration requirements are outlined in Part 3 of the Building Code and are usually left in the architect's design, while the force aspects are included in Part 4 of the Building Code and are the responsibility of the structural engineer. The structural engineer should be aware of several dimensional requirements and be able to question the architect if the proposed configuration does not match requirements. The dimensional items are shown in Table 11.1.

Table 11.1. A structural engineer's guide to dimensional requirements for guards and handrails

Dimension	Requirement
1070mm	Dimension to top of guard in exit ramps measured from top surface of ramp
1070mm	Dimension to top of guard from stair landings where the drop is less than 10m
1500mm	Dimension to top of guard from stair landings in exit stairs where the drop is greater than 10m
760mm	Height of guards in front of seats in assembly occupancy uses such as at the face of a theatre balcony
920mm	Height of guards at ends of aisles or base of stairs in assembly occupancy uses such as at the face of a theatre balcony
100mm	No sphere greater than 100mm shall pass through the deflected pickets of a guard. Deflection calculated using the 0.5kN picket load
140mm to 900mm	Between the height of 140mm to 900mm there shall be no element in the guard that assists climbing (this eliminates horizontal bars in this range)
920mm	Height of guards on stairs measured from stair nosing
30mm to 43mm	Minimum and maximum dimensions of a round handrail
865mm to 965mm	Maximum and minimum height of handrail in stair
1070mm	Height of handrails in landings may be 1070mm if guard is required. (aim here is to try to double up use of handrail as a guard)
50mm	Clearance between handrails and adjacent surfaces must be 50mm unless rough or abrasive in which case the clearance must be 60mm
2200mm	Maximum width of a stair before a centre handrail is required. (Stairs above this width require you to have a centre handrail and the handrail must follow the maximum dimensions of a handrail). Handrails on stairs that are wider than 2200mm must be at a maximum of 1650mm centre to centre

Guardrails are often built from round sections, in which case the designer has a choice of pipe sections or HSS. The pipe sections should be designed with a yield stress of 240MPa as this is the minimum yield for this material and verification of higher yield may not be reliable. The pipe sections have a lacquer applied which needs to be removed, preferably by Wheelabrator® abrasion prior to painting. HSS sections can also be called up. These sections do not have the same lacquer applied and are not only easier to paint but also have a higher yield stress. Due to availability, G40.21 HSS should not be used for round sections and ASTM A500 with a yield of 317MPa should be used. The section properties used should be those

corresponding to the ASTM A500 HSS sections, which takes account of the reduced wall thickness of those sections. The force requirements on the guards, pickets, and handrails are a mix of point loads and/or uniform loads. Where both a point load and a uniform load are given, both are to be considered with the element being designed to the worst of the two as shown in Table 11.2.

Table 11.2. Required loading for guard and handrail design

Location of guard / Type of element	Point Load	Uniform Load
Open viewing stands without fixed seats and for means of egress in grandstands, bleachers and arenas	N/A	3.0kN/m
Equipment platforms and stairs to these platforms where gathering of many people is improbable	1.0kN	N/A
Other guards not covered in the two categories above. (This is the most common type of guard in a building that is not a stadium.)	1.0kN	0.75kN/m
Pickets in a guard. The load may be considered over 100mm x 100mm	0.5kN	N/A
Handrail (This is from Part 3 of the Building Code.)	0.9kN	0.7kN/m
Downward load at top of guard (not considered to occur at same time as the outward load)	N/A	1.5kN/m
Vehicle guardrail in a building covered by NBCC design (e.g., parking facility). The point load is considered to occur at 500mm above the surface where the vehicle wheels are supported	22kN	N/A

Under NBCC 2015 a provision was introduced that permits the inward force on the guard to be half of the outward force, thus reflecting that a balcony guard is more likely to be pushed than pulled.

Pickets within the guard are required to resist a service level force of 0.5kN applied anywhere on the picket; however, under NBCC 2015 we are now required to consider the deflection of the picket when determining the picket spacing. The rule is that a 100mm diameter ball cannot be allowed to fit between the pickets in their deflected position. This is to prevent children and animals from forcing their way between the pickets of a guard and then getting stuck when they try to pull back.

The force on handrails is given in Part 3 of the Building Code as 0.7kN/m or a point load of 0.9kN. Handrails need to be graspable and for circular handrails the size range is 30mm minimum to 43mm maximum. This means that a 48mm HSS is not permitted to be a handrail. The height of the handrail has to be between 865mm and 965mm, which means that the handrail must be below the height of the top of the guard at 1040mm. Guards on stairs that are designed in accordance with the Code often have both a guardrail and a handrail as shown in the Figure 11.5. Stairs that are up to 1100mm wide can have handrails on one side only, if the stair is 1100mm to 2200mm wide it must have handrails on both sides of the stair or ramp. If the stair or ramp is over 2200mm wide there must be intermediate handrails, which will require intermediate posts.

EXAMPLE—SPACE HANDRAIL SUPPORTS

Determine the maximum distance between supports for a handrail made from round HSS with a diameter less than 43mm.

Solution:

We will use ASTM A500 sections, as G40.21 HSS sections are not easily available. Unfortunately the *CISC Handbook* does not list HSS sections that are smaller than 48mm. A 43mm maximum diameter allows us to use a 1-5/8-inch diameter (42.2mm) HSS but does not permit 1-3/4-inch (48.3mm) diameter HSS Sections. The only section with a 42.2mm outer diameter has a listed wall thickness of 3.58mm. As

with other ASTM sections, we are to determine the design properties using a wall thickness equal to 90% of the theoretical wall thickness. We are therefore wishing to find the moment resistance of a 42.2mm x 3.2mm HSS. The ASTM A500 sections have a yield of 317MPa. Table 11.3 shows the computed moment resistance for four round sections of various materials that comply with the maximum 43mm diameter for a handrail.

Table 11.3. Round members of various materials that can be used as handrails

Section	Remark	Yield (MPa)	Wall T for Calc.	Z x 10^3 mm^3	M_r (kNm)
HSS 42mm Diameter x 3.2mm	ASTM A500	317	3.20	4.88	1.39
Pipe 42mm Diameter x 3.6mm	Std. Wall	240	3.20	4.88	1.05
Stainless Steel 42.4mm x 2.5mm	AISI 304	215	2.5	3.99	0.77
Stainless Steel 42.4mm x 2.5mm	AISI 316	290	2.5	3.99	1.04

Having found the moment resistance, we can then determine the load. Typically the handrail will be continuous over several supports and we will take advantage of this by using some continuity in the moments as shown in Table 11.4.

Table 11.4. Load determined for various handrails

Load	Simple span	Continuous with all spans loaded	For continuous handrail
Point Load	PL/4	PL/8	PL/6
Uniform Load	$WL^2/8$	$WL^2/12$	$WL^2/10$

Let us use the section with the largest capacity (HSS 42mm Diameter x 3.2mm wall with a moment resistance of 1.39kNm. Looking at Table 11.2 we see the point load on a handrail is 0.9kN while the uniform load is 0.7kN/m. Load factor = 1.5 for live load.

For point load: $PL/6 = 0.9kN * 1.5 * L / 6 = 1.39kNm \rightarrow L = 6.1m$
For distributed load: $WL^2 / 10 = 0.7kN/m * 1.5 * L^2 / 10 = 1.39kNm \rightarrow L = 3.63m$ (Governs)
From a strength point of view this span is quite large and should be reviewed for deflection.

We might consider it a good idea to combine the handrail with the guard. This would work for the horizontal element but the vertical stanchions will be overloaded with any size that works for a handrail. If we have stanchions spaced at 1300mm, then the factored load at the top of the stanchion is:

- From the point load: 1.0kN * 1.5 = 1.5kN
- From the distributed load: 0.75kN/m * 1.3m * 1.5 = 1.46kN

The moment at the base of the stanchion is 1.5kN * 1.070m = 1.605kNm, which is not satisfied by any of the sections that have a diameter of less than 43mm. To address this problem, a 48mm diameter tube can be used for the stanchion which requires a more complex connection at the top because the handrail dimension cannot increase beyond 43mm. Alternatively an HSS 60x6.4 sleeve can be used at the base of the stanchion. The inner diameter of the HSS 60x6.4 is 47.2mm so the 42mm tube will fit in but needs to be wedged tight and seal-welded at the top. The maximum height of the 42mm tube to the point of the sleeve is 1.39kNm / 1.5kN = 0.927m.

One requirement that needs to be checked is the requirement that the handrail project beyond the end of the stair as shown in Figure 11.5. The handrail is required by Code to extend beyond the end of the

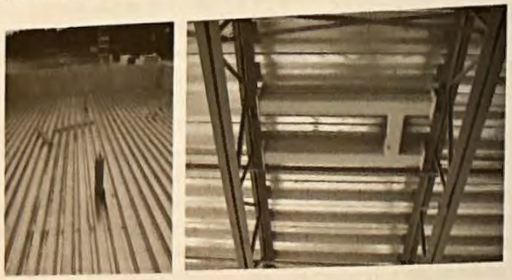
stair by 300mm and this is usually done by cantilevering the handrail. With our point load of 0.9kN and a projection of 300mm beyond the end of the stair or ramp (and assuming in this case that the last stanchion is positioned right at the base of the stair) we wind up with a factored moment of 0.9kN * 1.5 * 0.3m = 0.405kNm. This is satisfied by all the sections in Table 11.3, including those made from stainless steel.



Figure 11.5. Handrails at the base of a stair. Both the edge handrail and the centre handrail have Code mandated projection of 300mm beyond the end of the stair

11.3 FALL ARREST ANCHORS

Roof-mounted fall arrest anchors are frequently required where there is access to a roof for maintenance activities, including maintenance of rooftop mechanical equipment or access to window washing equipment. The challenge from the design engineer's standpoint is that the location of the anchors is determined by a speciality engineer who is usually retained after the structural drawings are tendered. While guardrails are designed as fall restraint – the aim being to prevent falling in the first place – fall arrest anchors are designed to withstand the impact of a falling worker sufficient to halt that person's fall. A fall arrest anchor must therefore be designed for an impact load. Fall arrest loads are specified by industrial regulations provided by provincial agencies such as WorkSafe BC. In British Columbia the fall arrest load is a factored load of 22kN (5000 lbs). Figure 11.6 shows roof and ceiling mounted fall arrest anchors. Figure 11.7 shows the structural drawings for the design of a roof anchor frame.



a) b)

Figure 11.6 a) Roof-mounted fall arrest anchor. b) Ceiling-mounted fall arrest anchor

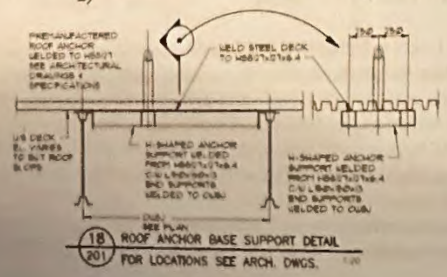


Figure 11.7 Roof anchor frame that can be used with OWSJ to produce only vertical loads on the truss

11.4 SUPPORT FOR STEEL STUDS AND GLAZING IN TALL WALLS

Areas of the building where the floor-to-floor height is greater than 5m will frequently require that the structural engineer put in support for studs or glazing. Examples of this are gyms where mid-height girts are used to reduce the stud size needed. Figure 11.8a shows a gym wall with a horizontal HSS at the mid-height level of the wall and above and below a strip window. The bay is complicated by being a brace bay. While the steel stud design is usually done by the speciality structural engineer and is not part of the base building engineer's scope, the elements that support the studs are structural items and must be shown on the structural drawings. The horizontal beams that are shown in Figure 11.8 were all fully designed by the base building engineer and shown on structural drawings. HSS beams are frequently used as the girts often support both vertical load from the dead load of the wall, which in the case of the wall in Figure 11.8 contained brick cladding, and lateral load due to wind and seismic loads. This means that the design engineer must compute the forces on the wall cladding from wind and seismic and the dead load of the wall. The HSS will therefore be under bi-axial bending and the tributary width for dead load may not be the same as the tributary width for lateral loads. Typically simple connections are used for ease of connection, as moment connections would be more bulky and may involve field welding.



a) b)

Figure 11.8. a) Extra framing in wall of gym. Note the horizontal HSS beam at the mid-point of the wall which reduces the size of the wall stud required and the horizontal HSS beams above and below the window. b) Airport terminals and other large public buildings will frequently have high glazing walls that require supports

Public buildings such as museums and airport terminals will often have tall feature glazed walls that require support, as the glazing mullions will not be able to span for the full height of the glazing. Figure 11.8b shows one such wall where the glass is supported by a mix of steel framing designed by the base building designer and the glazing mullions, which are designed by the curtain wall engineer. As the curtain wall designer will only be working on the job after the base building drawings are tendered it is necessary to state on the base building drawings what the assumptions are as to where the load of the glazing is picked up.

11.5 CANOPIES

Almost every building will have some sort of a canopy, and public buildings often have canopies that are particularly elegant features. As the structure is an integral part of the canopy, the architect will have a significant influence on the design and several iterations can usually be expected. Snow drift on canopies is an interesting challenge in Canada as the load on the canopy due to the drift is frequently several times the base uniform roof snow load. For example, going through the snow drift calculations for a building with $S_s = 1.8$ and $S_r = 0.3$, with a 60mx50m upper roof and a canopy 5m below the upper roof, gives a snow drift result on the canopy of almost 8kPa which is approximately 4.6 times the snow load on the upper roof. A snow load of 8kPa, requires almost 3m of 3kN/m^3 snow to be on the canopy, which is a doubtful situation. To get around this some engineers will limit the drift load on canopies to 2.5 times the roof snow, giving a canopy snow load of 4.8kPa in Vancouver (approximately 1.5m of snow at 3.0kN/m^3). This loading reflects the suggestion in the NBCC Commentaries that, for multi-level roofs, the depth of snow in the drift area should be evaluated to determine if it is reasonable. Canopies are often constructed using glass so as to provide more light to the region below. The glass should not be considered to be a diaphragm and structural elements should be analyzed and designed as if the glass provides load but no lateral resistance. Figure 11.9 shows a canopy for a retail store in Vancouver, in which the canopy support beams are plate sections that are considered to have no torsional resistance and $C_w = 0$. The load on these members comes from stools that support the glass, and the loading of the members above their centre of gravity needs to be considered in their design. The glass is not considered to be a diaphragm. As this canopy is in Vancouver, it also needs to be designed for seismic loads in all three directions – seismic loads parallel to the sidewalk will produce weak axis bending on the cantilevered beams.



a) b)

Figure 11.9. Structure supporting a glass canopy on a rainy day in Vancouver makes it easier to see both the glass and the rain shadow provided by the canopy. The canopy structure is formed from plates that cantilever from a member in the wall above the door

11.6 ELEVATORS, ESCALATORS, AND MOVING WALKWAYS

Mechanized vertical transportation in the form of elevators will be required in almost every multi-storey steel building. Many large public buildings will have escalators to provide continuous mechanically assisted movement between floors without interruptions. Large airports will often have moving walkways as international agreements limit the maximum walking distance between typical passenger stops to 300m and the distance taken up by the moving walkway does not count in this distance. From the structural engineer's standpoint the elevator will require the framing of holes through floors, and a pit of a certain size and depth at the base of the elevator. Elevators will also require an over-run at the top that may require a small "dog-house" on the roof containing the overrun of the elevator and possibly some of the elevator

equipment, including a hoist beam that is required to assist with the installation and maintenance of the elevator. Escalators (Figure 11.10a) will require framing for a pit at the bottom of the escalator and support beams to take the point load reactions from the escalator. Moving walkways (Figure 11.10b) will require pit framing at both ends of the moving walkway and need to accommodate point loads from the moving walkway at intermediate points. Considerable time is required on the part of the structural engineer to successfully coordinate the elevators, escalators, and moving walkways with the steel structure. Usually the structural steel is designed using preliminary shop drawings for the elevators and escalators which will contain load information, dimensional requirements, and locations of desired supports from equipment supplier. A second coordination cycle will be required when finalized shop drawings for the elevator equipment arrive, which is often after the structural steel is tendered.



a) b)

Figure 11.10. a) Extra framing for a pair of escalator pits. b) Framing for a pair of moving walkways in an airport. The lower section in the foreground is for the pit while the higher section is for the running section. The top surface of new installations will usually be flush with the floor

11.7 ART PROJECTS

Art projects are often installed in public buildings and the structural engineer will be expected to provide the structural design of the stability and support for these objects – often as an extra service. Alternatively another structural consultant will be brought in to work with the artist and act as an intermediary to the base building engineer. The extent of the involvement of the structural engineer will depend on the size of the piece and where it is to be located. At the lower end of the scale of involvement will be the review of the floor structure to take the weight of an art object. At the upper end of the scale the structural engineer will be called on to design the framework of the piece itself. Depending on your disposition, the designer may find working on art projects the most fun and interesting work or something to be dreaded. Each art project is unique and ingenuity is required to determine a solution that satisfies the structural Code requirements and forces of nature, while not offending the artist.

Art projects need to be designed for the following loads:

- Self-weight, including the effects of eccentricity – frequently sculptures are supported at a point eccentric to the centre of mass. Consider a statue of a sprinter that is in stride at the finish line, as shown in Figure 11.11, or a horse rearing on back legs. In these cases the addition of an extra column under the chest area of the sculpture while being the perfect structural solution is not an aesthetically acceptable option. The forces on the sculpture will depend on the weight of the piece, and while the artist may supply a weight, it is important to do independent calculations to verify this weight.
- Wind loading is determined conservatively, as the shape factors and drag coefficients are not easily available for most art projects. If in doubt, design it as if it were a billboard with the same maximum width and height as your sculpture. For some sculptures it may be desirable to consider

the piece as a series of rectangles. Tall slender sculptures may require a special wind study to evaluate the effects of vortex shedding. Even large art pieces should not be considered to shield an adjacent building from wind, as art tends to be transient and could be relocated within the life of the structure.

- Seismic loading on the sculpture is also important. Even in high seismic risk areas this is often not the governing criterion, as most artists want their sculptures to be as light as possible. Bronze sculptures are constructed with a moulding process that results in a hollow sculpture for all but the smallest of pieces. The determination of the seismic load requires a reasonable determination of the weight of the piece.
- A "yahoo factor" which considers one or two 1kN people (more for a larger sculpture) hanging off the sculpture in the most awkward way possible must also be considered. The "yahoo factor" should also include a lateral load of 1kN (service) for small pieces and perhaps larger for more significant pieces. The "yahoo factor" load need not be considered to act at the same time as wind or seismic loading, but does need to be considered in conjunction with the factored dead load of the structure. The "yahoo factor" is a judgment on the part of the designer and is not found in any Code and will often be the most significant load on the piece. It is better to use a larger "yahoo factor" load on the sculpture than be defending yourself from liabilities for injuries incurred by someone misusing the sculpture.

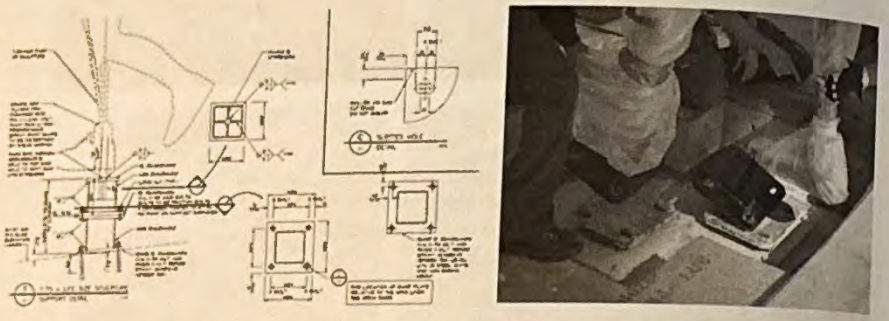


Figure 11.11. Base connection for bronze statue. The desire here is to create a two-part base where the bottom part can be installed first, allowing concrete work to finish before the sculpture is installed

An important aspect of design of the sculpture base connection is the decision on how much allowance for adjustment to provide. The support of the piece shown in Figure 11.12 allowed for adjustment in three directions by having slotted holes on the base in one direction and in the plate for the sculpture the holes were orthogonal, allowing the piece to have fine adjustment in the north-south and east-west directions as well as having a limited ability to rotate around a vertical axis. The addition of shims allows the piece some vertical adjustment in the upward direction only, as shims can be added but, once all the shims are removed, there is no ability to move further downward.

Most cast sculptures are struck from a structural bronze that has a melting point of approximately 950°C – considerably less than the 1370°C melting point of steel. The yield point and strength of the bronze will depend on the alloy used and, when checking the strength of individual pieces of a bronze sculpture, the designer will need to request material specifications for the bronze being used. For sculptures made from bronze that are anchored to steel supports there needs to be consideration of corrosion such that there are not dissimilar metals in contact. Neoprene gaskets between dissimilar metals and making sure that the contact locations are dry and with a limited oxygen supply all help to reduce the potential for corrosion.

CHAPTER 12
FABRICATION AND ERECTION OF STEEL PROJECTS

This chapter covers the designer's involvement with a structural steel project after the design is complete. This includes the review of shop drawings and the site review of the steel erection. We will also look at architecturally exposed steel which will require extra effort on the part of the designer if the appearance the architect is looking for is to be achieved.

12.1 SHOP DRAWING REVIEW

During design of a steel structure, the fabricator, manufacturer, supplier, and contractor will all produce shop drawings. Shop drawing review may seem to be a tedious task but it forms a very necessary step in the checks and balances that must occur in a structural steel design prior to its fabrication and erection.

Steel design results in a significant number of shop drawings that must be reviewed under tight schedule. On large steel projects, the number of shop drawings may be a hundred times the number of structural drawings for the project. The shop drawings are usually shipped to the design consultant with as many as six copies to be stamped and initialled and returned, all relating to one shop drawing, with comments needing to be transferred to all copies of that drawing.

The detailer is required by Clause 4.3.1 of S16 to provide the engineer with sketches showing his or her design intent for connections prior to the preparation of shop drawings. In practice this is seldom done except for larger projects. For larger projects the process of sketches in advance is to be recommended, as the fabricator will be more willing to change design-connection details if bundles of shop drawings have not already been produced. From the fabricator's point of view, showing early design sketches may avoid the delay caused by a cycle of "revise and resubmit" on the shop drawings.

Production of shop drawings by the fabricator will often result in requests for information (RFIs). These should be responded to with priority. Often the speed of response to the RFI will influence whether or not the information request turns into an extra charge. All responses to RFIs should go through the coordinating professional.

Slow review of shop drawings can result in construction delays and extra charges. For this reason accurate logs that track the arrival and departure of shop drawings should be kept. Transmittals should clearly show which drawings are being returned and the date of return. All shop drawings should go through the coordinating professional (usually the architect).

Immediately upon receipt of shop drawings, review them to ascertain that they are indeed part of your scope of review. Immediately return any drawings that are not structural in nature. Shop drawings that are for items not shown on your drawings should not be stamped or marked in any way that would indicate that you have reviewed these drawings.

The purpose of shop drawing review is to check for conformance with the contract documents, not to check for the accuracy or completion of dimensions and quantities.

Shop drawing review consists of the following checks:

1) Verify that the member size (e.g., W410x39) exceeds or agrees with the size shown on the structural drawings. Where size of elements has been increased by the fabricator, verify that the intent of structural and architectural drawings has been maintained. For example, if a W610x82 has been designed because its 178mm flange will fit in an 8-inch (203mm) stud wall, substitution of a W610x84 beam with a 226mm flange will violate one of the intents of the beam sizing. Also a heavier beam may not necessarily have an increased capacity; a W610x82 has a moment resistance of 683kNm while a W610x84 has a moment resistance of 627kNm ($F_y^2 = 345MPa$).
2) Check member length. The shop drawing review does not confirm dimensions and the shop drawing stamp will state this, but it is prudent to quickly verify that the member length is approximately as expected.
3) Review the proposed connection to determine if it meets the strength requirements.
4) Verify that the material grade satisfies the requirements on the drawings.
5) Review the camber of the beam to verify that it agrees with structural drawings. Floor beams should generally be cambered to give a flat floor after concrete is placed. This camber will be given as a function of length on the structural drawings (e.g., "Camber all simple span floor beams Span/360. Beams with spans less than 5m are to be uncambered except mill camber up").
6) Determine if any special details called for on structural drawings, including any mechanical holes or brackets shown on sections on the structural drawings, have been included correctly in the shop drawings.
7) Check for any holes that have been added because the fabricator has referred to mechanical or electrical drawings. The resulting holes may violate the intent of your design.

The following items are generally not reviewed by the structural engineer:

1) Details to do with painting shown on architectural drawings.
2) Exact dimensions of the beam (shop-drawing stamp should say that this is not being reviewed by the structural engineer).
3) Quantities on the bill of materials.

Shop drawings should be stamped with a company stamp that includes the date of review and gives appropriate disclaimers with wording suggested by your liability insurance provider. The disclaimer will include words to the effect of not being responsible for dimensions.

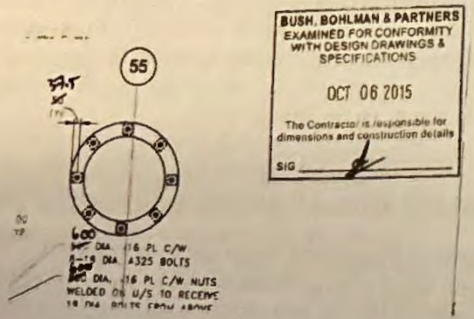


Figure 12.1. Structural engineering firm stamp showing date of review and dimensions disclaimer

The shop drawings are stamped "Reviewed" and the word "Approved" is never used – make sure this also applies to your returning transmittal.

Clause 4.3.3 of S16 requires that erection diagrams be submitted to the designer for review. Often referred to as "E-Diagrams," they show the general arrangement of beams and provide a key plan for locating the beam mark numbers. They may also provide information about construction sequencing or temporary bracing requirements. An example of a steel "E-Diagram" and the referenced shop drawing is shown in Figure 12.2.

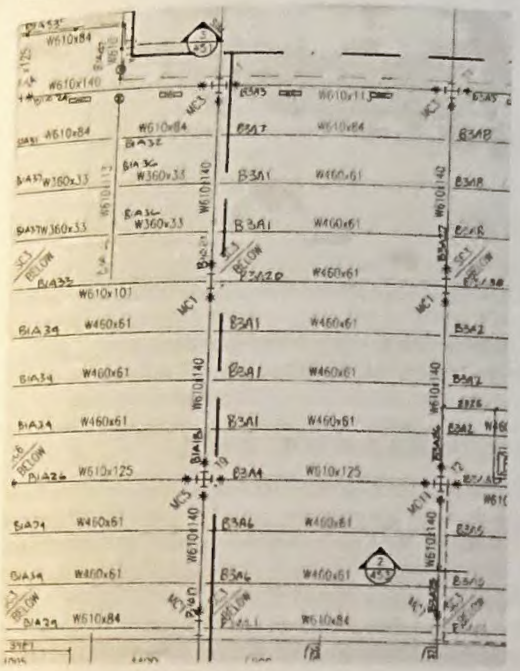


Figure 12.2. Sample erection shop drawing

An E- Diagram as shown Figure 12.2 gives the location of the beam shown in the shop drawing, as shown in Figure 12.3. In this case B3A1 can be seen in five locations.

The shop drawing shows the beam with exaggerated scale such that bolt holes, duct holes, copes, and other details can be seen easily by the fabrication shop. As a result, dimensions on shop drawings must never be determined by scaling.

The shop drawing will be checked for

- correct beam size (W460x61)
- correct steel grade
- number of bolts satisfies connection requirements
- conduit hole (as shown on structural drawings)
- camber

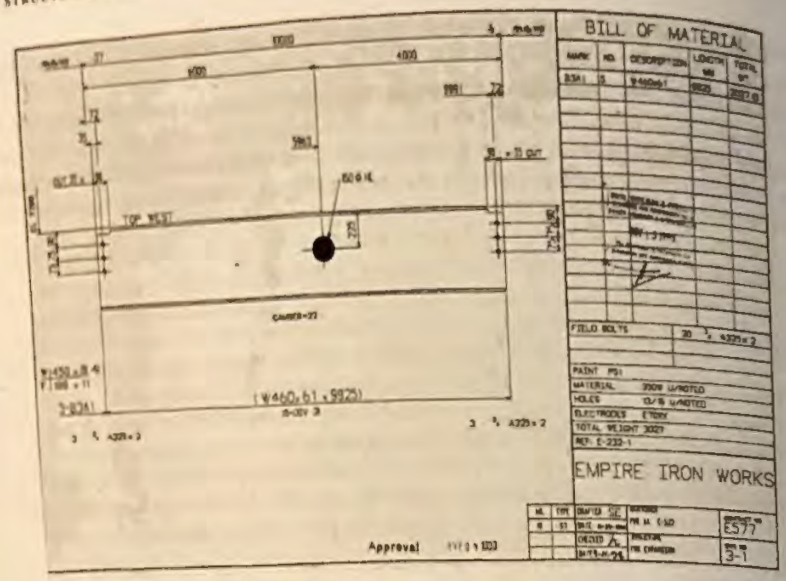


Figure 12.3. Sample shop drawing corresponding to E-Diagram
Source: Reproduced courtesy Empire Iron Works

12.2 SHOP DRAWING PRODUCTION USING 3-D SOFTWARE

Over the past decade the use of computer software to produce shop drawings has become more common to the point where it is now very rare to see shop drawings not prepared using 3-D software. Programs such as X-Steel are used to produce a three-dimensional model of the project that includes all connections. Steel detailing programs often cost many times what the structural analysis software for the project cost, and considerable training is required to be a productive operator. However, the 3-D model can be a very useful tool in determining the location of conflicts between the steel before it is fabricated.

A growing trend is to have the structural engineer much more involved with the fabrication process in a manner that blurs the lines between the two parties. It is now more common for the structural engineer to transfer information to the fabricator electronically, including 2-D and 3-D models that can be read directly into the fabricator's detailing model.

There is a growing trend in the design and construction industry to move to a Building Information Modelling (BIM) standard of project delivery. In a BIM project, the architect and the consultants on the project document their design through a 3-D model that is produced through specialized programs that include AutoDesk's Revit. The resulting model builds the structure in a virtual world and identifies many of the conflicts that may occur between structural steel elements and other services in the building. At about the same time the transfer of BIM models to the fabricator is occurring, it is not uncommon for the steel fabricator to build a completely different 3-D model in his or her detailing software. This seems wasteful but is a function of several factors, including the compatibility between the detailing software (often Tekla-based) and the BIM model (often Revit-based). There is also concern on the part of the fabricator that the model produced by the design engineer may not be as accurate as is needed for steel detailing. A sample 3-D drawing is shown in Figure 12.4.

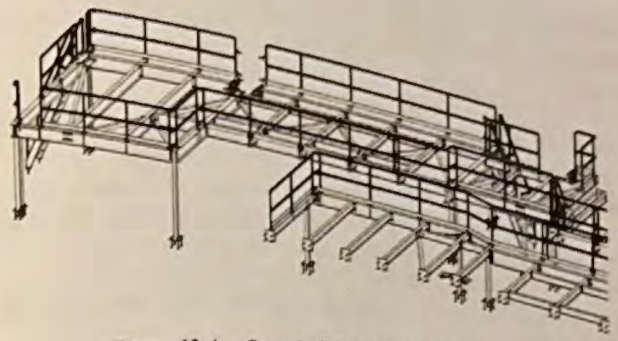


Figure 12.4. Sample 3-D or Tekla drawing
Source: Scott Detailing

12.3 SITE REVIEW OF STRUCTURAL STEEL CONSTRUCTION

Much of the field-services time spent by the designer during the construction of a steel building is in the review of shop drawings. Consequently, the number of site visits for the designer during the erection of a structural steel building is much less than for a similar sized concrete building.

Site review by the designer consists of the following:

1) Determine that the members called for on the drawings have been placed in the field. This inspection is generally quite rapid as missing members will usually be spotted quickly.
2) Review to see that all trimmer beams around openings have been placed.
3) Review beam connections to see that all bolts have been placed. Again, this is a very rapid inspection. Missing bolts are obvious, although connections might be obscured by other building elements.
4) Review to see that all puddle welding or mechanical connection of steel roof and floor deck has been placed. Particular attention should be paid to making certain that welding or mechanical connection between shear lugs and deck has occurred and between beams running parallel to the deck span.
5) Visit the site prior to placing concrete on metal deck to verify that reinforcing steel has been placed.
6) Spot-check to confirm that grout has been placed under columns. The grout pack should be placed under the column prior to any significant load being added to the column, including the weight of any concrete on the metal deck. The grout pack will only be placed after the column is shimmed up.
7) Review any inspection reports (usually welding reports) prepared by inspection agencies that visit the fabrication plant and site of the erection. Of particular concern is the resolution of any deficiencies that are found by the testing agency.
8) Based on the site review, approve progress-payment draws.

The steel fabricator will follow quality control procedures and hire a welding engineer and contract with a testing agency for review of some of the shop and field procedures. On larger jobs the designer will often meet these engineers and technicians on site and, as a first step, welds and connections that look suspicious should be discussed with these professionals.

Visual inspection of welding by a qualified inspection agency can find the following issues:

- Incomplete weld segments
- Irregular profile
- Excess undercut
- Insufficient throat
- Excess slag

- Unfilled craters
- Excess convexity
- Burn-through

Many structural engineers would have difficulty distinguishing one type of fault from another, but if the weld looks ugly you probably have found at least one of these problems.

Mill certificates for steel sections and bolts should be available if requested from the steel fabricator and each piece of steel should be referenced to its mill certificate. However, the review of the mill certificates is usually performed by testing agencies for the owner and contractor.

Final steps in the site review of structural steel construction include:

1) Prepare dated deficiency lists for work not completed or not completed to the satisfaction of the design team.
2) Provide follow-up site visits to review any remedial work before these areas are covered by building finishes.
3) Sign and seal the final letter of assurance.



Figure 12.5. Active site during construction

12.4 GENERAL NOTES AND SPECIFICATIONS

The design drawings for a project containing structural steel will require general notes to cover those items that are needed by the Authority Having Jurisdiction, as well as by those building the structure and those reviewing it or modifying it later. This information is essential for getting a building permit, for making sure the construction of the building is carried out with the correct methods and materials, and for determining the capacity of the building to carry extra loads in the future. This information would include those items required by S16 Clause 4.2.2 but not shown otherwise on the drawings, including the following:

1) State the design standard used (S16), including the edition, and the appropriate edition of the Building Code. It is important that these references be up-to-date. The designer should also be familiar with the referenced standard and have a copy for reference in the office. It is important that the correct Building Code be referenced. For example, submitting a job to the City of Vancouver stating that the building was designed to the National Building Code of Canada is a good flag to the City that additional scrutiny of the drawings would be appropriate (the City expects drawings to be submitted using the Vancouver Building Bylaw).

2) Specify the material or product standards. Again it is important that these references be up-to-date. The designer should also be familiar with the referenced standard and have a copy for reference in the office.
3) Identify the design criteria for wind, seismic and special loads. These loads should be the appropriate ones for the site and will need to be modified from one job to the next.
4) Specify live loads and superimposed dead loads for floors and platforms. These also need to be modified for the job at hand – don't give live load in classrooms if the notes are for a building that does not have classrooms.
5) Identify the structural system used for the seismic design, including the appropriate R_dR_o values, as well as the information required to reproduce the seismic forces used by the designer, including the $S_a(0.2)$ value, the Importance Category, and the Site Class.
6) State the requirements of roof and floor diaphragms, including connection requirements if not shown elsewhere on the drawings.
7) Specify the required camber of beams, girders, and trusses.
8) State the types of bolts and the pre-tensioning requirements. Also state the designation of joints as bearing or slip-critical.
9) Describe the foundation information used in the design, including a reference to the geo-technical report that was used in the preparation of the foundation design. If spread footings are used, then the allowable and factored bearing resistance should be given; if piles are used, then include the design capacity of the piles.

Other items called for in Clause 4.2.2, such as the details for seismic connections, will usually be shown as details on the drawings rather than in note form in the general notes. As there is a tendency to copy the general notes from one project to the next, the designer should make sure that the general notes are appropriate for the project at hand. As an example, the notes on increasing the capacity of the bottom chord on trusses supporting basketball hoops may not be appropriate in the design drawings of a hospital. Similarly the notes copied from a previous job may be missing important items that are required on the job at hand but were not required on the previous job. Some consultants will use boilerplate general notes that have two to three sheets of notes and typical details that may or may not apply to the job at hand but are there "just in case." Excessive general notes will tend to cause confusion, especially if the job is small, and may even result in them remaining unread.

Specifications will often state much of the information that is stated in the general notes but in a more formalized manner. The designer should make sure that the items stated in the specifications do not conflict with the material shown on the drawings. In the case of conflict between drawings and specifications, the specifications govern. Wherever possible, especially for small jobs, including minor renovations to existing structures, technical specifications should be avoided. This has the following advantages:

1) There will never be conflict between the specifications and the drawings if specifications do not exist. This might seem obvious but it is surprising how often items on drawings will get changed and updated without a corresponding change occurring in the specifications.
2) If the items desired for construction are specified in the general notes and drawings, then the information will be available in the field to those doing the construction and trying to verify that it has been done in accordance with the contract documents. Drawings will be on the site and in the hands of the field inspection team, whereas the specifications seldom leave the design office or the site trailer.
3) The drawings will be stored by the designer, the Authority Having Jurisdiction, and often the owner, whereas specifications will frequently disappear following the completion of the project. The engineers trying to evaluate or renovate a building several years after its construction will be frustrated if the necessary structural design information is contained in the now-missing specifications. Notes that read "For typical wall reinforcing, see specifications" make it very difficult to determine the capacity of the walls if the specifications have long ago been lost. Typical details can alternatively be included on the drawings.

12.5 CHANGES TO DRAWINGS

A normal structural steel project goes through several stages from conceptual design and design development to working drawings and tender, then to shop drawings, fabrication, and erection. At the preliminary phases the project will be very fluid and changes in structural sizes and configuration are common. As the working drawings progress, the design solidifies to the point where it should be frozen at tender. To facilitate changes during the development of the drawings, in general information should be called up in one location only. If the same information appears in multiple places, it is likely that one or more of these will be overlooked if a change is made.

Costs of changes will increase as the project advances. Changes in the structural design during the conceptual phase are expected and come with no costs to the project. Changes during tender are less expected and come with costs. Changes during shop drawing preparation and review will be even more expensive and are less tolerated by project managers and owners. Changes during or after fabrication can be very expensive and will usually involve significant loss of credibility for the design team. Designers should be careful in situations where the structure is tendered well before the architectural design is complete, as this will increase the probability of changes in the structural steel during tender and shop drawings. Unfortunately, the pre-tendering of structural steel is becoming the norm in the present fast-paced construction environment.

12.6 OTHER REFERENCED STANDARDS AND CANADIAN WELDING BUREAU CERTIFIED FABRICATION PLANTS

Two standards are very important to those who are carrying out the welding but may not be so well known to the design engineer. These include the following:

1) W47.1 Certification of companies for fusion welding of steel
2) W59 Welded steel construction (metal arc welding)

The W59 standards establish the quality requirements for the welds, while W47.1 sets the standards that the welding companies must adhere to if they are to be certified.

It is usual that steel fabrication plants be certified by the Canadian Welding Bureau, to establish a level of checking and quality control in the plant, and that the welders are certified for the welds that they are performing. Drawings and specifications should require that the fabricator and his or her plant and welders be Canadian Welding Bureau (CWB) certified.

This requirement was emphasized by the Association of Professional Engineers and Geoscientists of British Columbia in May 2008 in the article "Structural Engineers Required to Check Qualifications of Welded Steel Fabricators and Erectors," published in *Connections*, vol. 4, no. 5, and available on the APEG website. The article said, in part, as follows:

> Recently, a structural engineer of record in Alberta issued a letter of certification indicating that the structural steel construction on a project was in compliance with the Alberta Building Code (ABC); however, the firm that had done the welding for the structure had not been certified by the CWB. The engineer was subsequently reprimanded. Structural Engineers of Record involved in buildings constructed with steel are responsible for verifying that fabricators and erectors are certified by the Canadian Welding Bureau if the project is to comply with the Alberta Building Code.
>
> A similar requirement is contained in the British Columbia Building Code (BCBC) 2006, Division B, Clause 4.3.4.1(1), which refers to CAN/CSA-S16-Cl 24.3. Engineers involved in project approval, structural design, review of shop drawings, and contractors' submittals, and field review or inspection should be familiar with all aspects of compliance with the Building Code, including CWB certification. The requirement for CWB certification applies to a fabricator, erector, or manufacturer of structural elements of any building where welding is involved, including pre-engineered steel buildings.
>
> Design, fabrication, and erection of structural steel buildings must conform to the requirements of Part 4 of the BCBC 2006, "Structural Design." Materials must conform to Section 4.3 "Design Requirements for Structural Materials." Specifically, for steel, Section 4.3.4.1 "Design Basis for Structural Steel" requires compliance with CAN/CSA–S16.1 "Limit States Design of Steel Structures" and Section 4.3.4.2 "Design Basis for Cold Formed Steel" requires compliance with CSA S136, "Cold Formed Steel Structural Members." Qualification for fabricators and erectors of welded construction is found in Clause 24.3 of this CSA Standard, which states: "Fabricators and erectors responsible for making welds for structures fabricated or erected under this Standard shall be certified by the Canadian Welding Bureau to the requirements of CSA.

There are three levels of certification for plants under the CWB as summarized in Table 12.1. For steel structures the fundamental difference is whether a qualified welding engineer is retained on staff or on retainer.

Table 12.1. Certification requirements for fabricators

Requirements for Certification	Division 1	Division 2	Division 3
Qualified Welders	Yes	Yes	Yes
Qualified Welding Procedures	Yes	Yes	Yes
Qualified Welding Supervisor	Yes	Yes	Yes
Qualified Welding Engineer	Yes - Employed full time	Yes - On Retainer	No

The level of certification for Division 1 or 2 means that those certified to this level are more organized and established than those certified to Division 3. Plants should be certified to Level 1 or 2, and CWB recommends the following wording on drawings and specifications:

> Welding shall only be undertaken by a company Certified by Canadian Welding Bureau to the requirements of CSA Standard W47.1 (latest edition) "Certification of Companies for Fusion Welding of Steel" in Division 1 or 2. All welding shall be done in accordance with and meet the quality requirements of CSA Standard W59 (latest edition). All welders employed by the company shall hold valid qualifications issued by the Canadian Welding Bureau.

12.7 ARCHITECTURALLY EXPOSED STRUCTURAL STEEL

Elegant and dramatic exposed structural steel is a significant feature of many modern buildings. While the designer tends to look at the steel from a structural standpoint, the architect will often look very closely at the detailing from an aesthetic point of view. The CISC and AISC have developed guidelines for architecturally exposed structural steel (AESS) that will help with this process. Details of the connections should be carefully considered with the architect and noted on drawings where the requirements exceed normal practice such that they can be priced appropriately by the fabricator.

Categories proposed by CISC for AESS are given in Appendix I of the CISC Code of Standard Practice (Part 7 of the *CISC Handbook*) and reproduced as follows:

Category AESS 1: Basic Elements

Suitable for "basic" elements which require enhanced workmanship.

Category AESS 2: Feature Elements Viewed at a distance >6m

Suitable for "feature" elements viewed at a distance greater than six meters. This process involves basically good fabrication practices with enhanced treatment of weld, connection and fabrication detail, tolerances for gaps, and copes.

Category AESS 3: Feature Elements Viewed at a distance = <6m

Suitable for "feature" elements – where the designer is comfortable allowing the viewer to see the art of metalworking. Welds are generally smooth but visible; some grind marks are acceptable. Tolerances are tighter than normal standards. The finished product would normally be viewed from distances closer than six meters and is frequently subject to touching by the public.

Category AESS 4: Showcase Elements

This category would be suitable for "showcase or dominate" elements – used where the designer intends that the form is the only feature showing in an element. All welds are ground and filled edges are ground square and true. All surfaces are sanded / filled. Tolerances of fabricated forms are to be more stringent – generally one half of standard tolerance. All surfaces to be "glove" smooth.

Category AESS C: Custom Elements

Suitable for elements which require a different set of characteristics than specified in Categories 1, 2, 3 or 4.

Applying aesthetic rules to steel construction will add to the cost and should be clearly noted on the drawings. AESS is not something that should appear suddenly during shop drawing mark-up.

12.8 PAINTING

The architect usually specifies painting of structural steel; however, some guidance can be found in Clause 28 of S16. Pay special attention to areas such as those to be welded or the faying surfaces of slip-critical joints as these areas must be kept free of both paint and primer. Typical steel primer is not a permanent finish, and when exposed to weather without a finish coat for periods longer than six months, cannot be relied on to protect against rusting. Care must also be taken to have the architect verify that the primer is compatible with the proposed fireproofing methods, which are also specified by the architect.

The use of lead in paint was very common until the 1980s and poses significant cost issues due to environmental concerns if it needs to be removed as part of a renovation. Structural drawings and specifications should state that all paint and primer is to be free of lead and mercury.

12.9 GALVANIZING

Galvanizing is a great solution for protecting small pieces of steel from corrosion but sometimes an architect will suggest using galvanizing for his or her AESS project, thinking that shiny galvanizing found on some new garbage cans will be reproduced in the project. The structural designer needs to be pro-active on this one and should steer the architect away from this solution as the end product will often, if not always, disappoint. Try suggesting silver paint in lieu of galvanizing. Galvanized steel should not be used for AESS for the following reasons:

- The galvanizing is never as uniform as the architect would like it to be. In some places the galvanized surface may appear milky with a white powdery deposit. In other places the surface will be shiny, while seemingly similar places will be low lustre or dull finish.
- Even uniformly galvanized pieces can change appearance during storage. Pieces exposed to weather will look different from those not exposed.
- The material will lose its lustre over a short enough period of time (two years or less, depending on the exposure conditions) and this will disappoint the architect.
- The galvanizing process involves dipping the completed fabricated pieces into a hot bath of liquid zinc. This can cause the piece to warp, which will be unacceptable to the architect.
- The "baths" that the piece is submerged in are of limited size and the design must be fabricated from pieces that fit, potentially requiring connections that may not be part of the architect's concept.
- Welding to galvanized items requires removal of the galvanizing prior to welding (the fumes given off by the welding are toxic and the galvanizing material is also not good for weld quality). Welds must be touched up with paints such as "Galvacon," which seldom match the galvanizing of the area adjacent to the weld.
- The architect will inevitably be unhappy with the final result and will decide that it is so unacceptable that the AESS needs to be painted, which will result in extra charges that the owner won't want to pay.
- Some studies have shown that hot dip galvanizing can make HSS sections brittle and lead to cracking.

CHAPTER 13
INTRODUCTION TO SEISMIC DESIGN

Application of seismic design principles is fundamental to the successful design of all buildings in British Columbia as well as many other parts of Canada. This chapter concentrates on the fundamentals of seismic design including the basic philosophy. The notes will then review the calculation of lateral forces for seismic design using the static procedures of NBCC 2015 for low-rise buildings. Finally these forces will be applied to a simple building.

13.1 BACKGROUND TO SEISMOLOGY IN BRITISH COLUMBIA

British Columbia is adjacent to several intersecting tectonic plates, resulting in an active seismic environment. Minor earthquakes are measured on a daily basis, and the province is home to one of the largest earthquakes ever recorded: a magnitude 8.1 event that occurred in the Queen Charlotte Islands in 1949. Many of the earthquakes that occur in British Columbia are not felt; nevertheless, the significant probability of a major event occurring in a populated area has made seismic loading the governing lateral load for most low-rise buildings. Smaller "crustal" events that have occurred in the past in British Columbia and Washington State are:

- December 15, 1872, magnitude 7.4 event near Seattle
- December 6, 1918, magnitude 6.9 event Vancouver Island
- June 23, 1946, magnitude 7.3 event near Campbell River
- August 22, 1949, magnitude 8.1 event in Queen Charlottes
- April 29, 1965, magnitude 6.5 event north of Tacoma, Washington
- February 28, 2001, magnitude 6.8 event near Olympia, Washington

All of these events are larger than the magnitude 6.3 event that caused extensive damage in Christchurch, New Zealand, on February 22, 2011.

For most of coastal British Columbia, there is a threat of the "Great Subduction Event." This event occurs every 300 to 500 years, and the most recent was January 26, 1700. The event is similar to at least four great seduction events that occurred in the past 50 years with significant loss of life:

- Boxing Day, 2004, magnitude between 9.1 and 9.3 in Sumatra
- May 22, 1960, magnitude 9.5 event in Chile
- February 27, 2010, magnitude 8.8 event in Chile
- March 11, 2011, magnitude 9.0 event off Sendai, Japan

The probability that the subduction event will occur increases as we pass the 300-year mark since the last event. The probability that the event will occur in the next 50 years is 1 in 8, while in the next 100 years it is 1 in 4. These probabilities mean that there is a very real chance of the effects of this event acting on the buildings we design today.

13.2 FUNDAMENTALS OF SEISMIC DESIGN

Seismic design is fundamentally different from most other loads in that it is a dynamic loading that depends, at least in part, on the properties of the structure. Seismic loading also has considerably more uncertainty when compared to other types of loading.

There are three basic rules for seismic design:

1) Ductility
2) Capacity Design
3) Detailing

1) ***Ductility*** – The structure must have elements that can accept yielding and dissipate energy. When computing forces, our Building Code includes a ductility factors R_d and R_o that reduces the code forces to a more manageable value. But to take advantage of this, there must be a mechanism that allows yielding to occur. The fundamental diagram to illustrate this principle is shown in Figure 13.1.

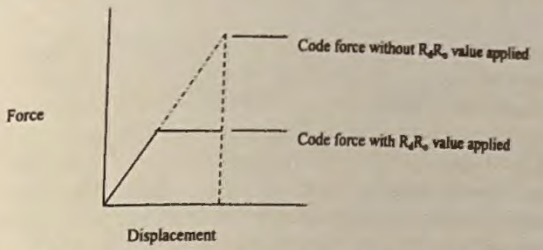


Figure 13.1. Equal displacement theory which shows that structures with or without yielding will displace equally. The desire is to have a yielding mechanism to get to that displacement while keeping the forces in check

The principle behind Figure 13.1 is that the structure will displace the same amount under seismic loading regardless of the amount of yielding in the structure. This is known as the equal displacement principle. If no yielding is allowed to occur, then the force will increase to the amount that would be found with the code forces calculated using a ductility factor (R_dR_o) of 1.0. You cannot use the reduced forces with the R_dR_o-value applied unless yielding is allowed to occur.

If there is excess capacity in the seismic system before yielding occurs, then the behaviour will be as shown in Figure 13.2.

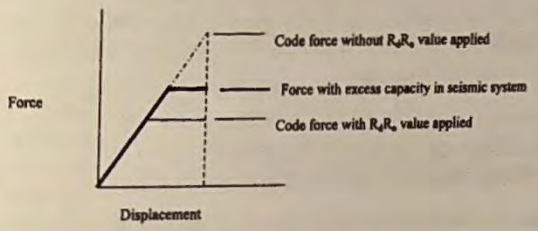


Figure 13.2. Effect of overstrength on the forces and displacement of the building. While forces are higher the building displaces the same amount

2) ***Capacity Design*** – Define a location where yielding occurs and design such that yielding occurs at the required force levels. This is one time that stronger is not better. Design all other elements that link to the yielding element to be stronger than the yielding element. The yielding element is to act as a fuse protecting the more brittle and less detailed sections of the structure. Under

capacity design the forces in elements in the structure are determined from the yielding element; the code forces that defined the minimum point of yielding no longer matter. A diagram showing how a yielding element protects the structure is shown in Figure 13.3.

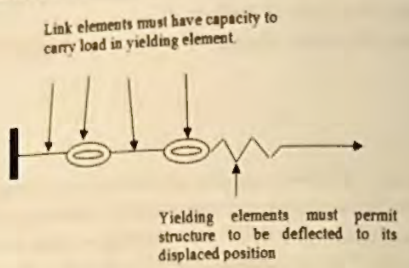


Figure 13.3. How a yielding element protects the structure

3) ***Detailing*** – Much of the art of seismic design is in the detailing and tying everything together such that inertia forces from any part of the building have a path to the seismic resisting elements that can carry the forces out of the building. A building structure is made up of a series of systems: the vertical load carrying system, the diaphragm system, the bracing system and the foundation system. While good seismic detailing takes care of each system, it is equally important to detail the connections between adjoining systems. It is also very desirable to make the load paths as direct as possible. Complex load paths will have more opportunities to illustrate any weakness in seismic detailing. Past earthquakes have shown that seismic loading is ruthless in finding any weaknesses in the detailing or load paths of the lateral system.

An important part of seismic detailing is to try to make the building "regular." Following the February 22, 2011 earthquake in Christchurch of magnitude 6.3 which resulted in several collapsed buildings and deaths, the engineering association in New Zealand (IPNEZ) issued a statement which contained an excellent summation of this philosophy:

> A number of experienced structural engineers have observed that buildings with well-conceived and simple structural systems with minimal irregularities have exhibited superior performance to those which may have only nominally or theoretically met codified requirements.

As many of the buildings that collapsed in this earthquake were reasonably modern the New Zealand government has conducted a Royal Commission to determine the cause of the failures and determine if the current seismic codes were sufficient. The result was mandatory upgrading of seismically deficient buildings and a strengthened building code.

13.3 CALCULATION OF SEISMIC FORCES USING CODE STATIC PROCEDURES

The seismic loading on a building comes from movement, as well as from imposed deformations and the effects of inertia forces. The load on the building will be a function of the dynamic properties of the structure interacting with the effects of the seismic event at the site.

While seismic forces are dynamic and complex in nature, it is common for the seismic design of low-rise buildings to be performed using the static procedures outlined in Clause 4.1.8 of the Building Code. The equation for computing the base shear on a building is as follow:

$$V = \frac{S(T_a)M_v I_E W}{(R_d R_o)}$$

Two equations limit the upper level of seismic force on a building with a seismic system having an R_d of 1.5 or greater and not located on site Class F. We can limit the force that is calculated in the base shear equation to the larger of the following two equations:

$$V = \frac{2}{3} * \frac{S(0.2) * I_E W}{(R_d R_o)}$$

$$V = \frac{S(0.5) I_E W}{(R_d R_o)}$$

There is a also an equation that provides a "floor" for the base shear – requiring that the base shear exceed a minimum value that depends on the type of structure. This lower limiting value applies to moment-frame and brace-bay buildings that have a period greater than 2 seconds and to wall structures that have a period greater than 4 seconds. For low-rise buildings less than 10 storeys, these lower bound equations will not enter into consideration.

Where:

$S(T)$ = $F(0.2)S_a(0.2)$ or $F(0.5)S_a(0.5)$ whichever is larger for $T \leq 0.2s$
= $F(0.5)S_a(0.5)$ for $T = 0.5s$
= $F(1.0)S_a(1.0)$ for $T = 1.0s$
= $F(2.0)S_a(2.0)$ for $T = 2.0s$
= $F(5.0)S_a(5.0)$ for $T = 5.0s$
= $F(10.0)S_a(10.0)$ for $T \geq 10s$

$S(0.2)$ = Site spectrum acceleration at 0.2 seconds determined in accordance with code established procedures that take into account the firm ground acceleration expected and the soil at the site. These values can be found in Appendix C of the Building Code. Values for the seismic coefficients can also be found from the earthquakes Canada website (at no charge) by inputting latitude and longitude of the site. For Vancouver, use $S(0.2) = 0.848$. The values are given by the Geological Survey of Canada and are the mean values with an expectation of being exceeded by 2% in 50 years.

I_E = Importance factor =

- 1.0 for normal buildings
- 1.3 for *High Importance* buildings including elementary, middle, and high schools, community centres, and buildings containing large amounts of toxic material.
- 1.5 for *Post Disaster* facilities such as hospitals, fire and ambulance stations, however, both seniors housing and buildings such as cancer clinics are regarded as being normal importance.

There is a *High Importance* rating for schools for three reasons:

1) It is considered necessary to protect the school's young and innocent occupants. Consequently it is desirable to have a lower probability of collapse.
2) The children have less ability to rescue themselves from a damaged structure so it is desirable to reduce the probability of damage.
3) Schools are often used as gathering places and emergency shelters in the community and as such it is necessary that they be more likely to function after a major event.

Note that these three reasons only apply to schools housing kindergarten to grade 12 students. As far as the Building Code is concerned, university and college buildings are not considered to be schools. Under Canadian codes, buildings that have high occupancy from gathering of many people, such as theatres, stadiums or airports, are not rated as *High Importance*. These buildings are regarded as being normal importance structures.

W = Weight of the structure including partition load and 25% of snow load. Use $C_a = 1.0$, $C_v = 0.8$ (no drifting) when evaluating snow load for seismic weight. Items that are permanent in the building such as roofing material and mechanical equipment should also be included in the weight. Where there is storage loading 60% of the storage loading should be included and the full weight of liquids in tanks. It is not necessary to consider the full partition load, live load and snow load when computing the weight that is used for calculating seismic shears. NBCC recognizes that the minimum partition weight of 1.0kPa required vertical load design is quite conservative when it comes to seismic design and the definition for W in Clause 4.1.8.2 states that "the minimum partition load as defined in sentence 4.1.4.1.(3) need not exceed 0.5kPa." As there is a limited probability that the full snow will be on the roof at the time as the design level earthquake, we only need to include 25% snow load in our computation of W; however, we are required to include the importance factor I_E so the snow load used for the weight calculation on *High Importance* and *Post Disaster* roofs will be slightly higher than for normal importance buildings.

R_d = Ductility related force modification factor reflecting the capability of the structure to dissipate energy through inelastic behaviour. The values for R_d are given in the Building Code and to use the value desired it is necessary to follow the detailing rules in the appropriate code. For concrete systems this is determined by following **ALL** the detailing requirements outlined for the appropriate system in Clause 21 of CSA A23.3. For a steel system we must follow **ALL** the requirements outlined in Clause 27 of S16. The R_d-value represents the amount of ductile behaviour that can be expected from the system and for steel seismic systems the value of R_d ranges from 5.0 for very ductile systems to 1.5 for systems that are not detailed for ductility. We will discuss the Lateral Force Resisting System (LFRS) more in Chapter 16.

R_o = The overstrength related force modification factor accounting for the dependable portion of the reserve strength in a structure designed to the provisions of the steel standard. The overstrength factor is prescribed in the Code but it is a function of several factors as follows:

$$R_o = R_{size} * R_{\varnothing} * R_{yield} * R_{sh} * R_{mech}$$

Where:

R_{size} = Rounding of sizes and dimension (need 3.5 bars place 4 would be an example).

$R_{\varnothing}$ = Difference between nominal and factored resistance (*i.e.*, $1/\varnothing$)

R_{yield} = Ratio of actual "yield" to minimum specified yield (Specify Fy = 345MPa but actually comes out higher.)

R_{sh} = Overstrength due to strain hardening.

R_{mech} = Overstrength arising from mobilizing full capacity of structure (getting a collapse mechanism).

13.4 NBCC 2015 BASE SHEAR USING STATIC PROCEDURES

Three important changes in the static Code procedure were introduced with NBCC 2015, all of which have an effect that can raise the required seismic shear on the building. The first of these changes is limiting the plateau of the cut-off equation to a maximum of 0.5 seconds. The cut off equation is only intended to apply to structures that have a period under 0.5 seconds and under NBCC 2010 the two-thirds cut-off would often extend to 1 second for structures on soft soils and this was not the intent of the cap. The Mexico City earthquake of 1985 showed the catastrophic damage that can occur to buildings on soft soils in the 1 second period range. For this reason the equation:

$$V = \frac{S(0.5) I_E W}{(R_d R_o)}$$

is introduced in NBCC 2015 to prevent the plateau exteading beyond 0.5 seconds as shown in the cartoon design spectrums shown in Figure 13.4. The effect of this change is dependent on the location and the site class. Sites located on rock or firm ground will be unaffected while those sites located on soft soils in high seismic risk areas will be affected most.

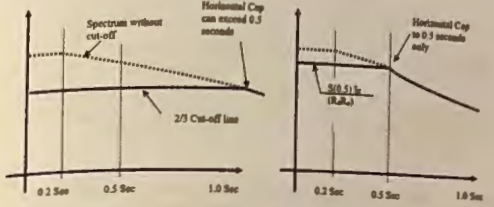


Figure 13.4. Cartoon spectrums showing how the limiting value of the cut off shear value increases under NBCC 2015 so that the cut-off plateau does not extend beyond 0.5 seconds

A second significant change with NBCC 2015 is but different on the effect on the calculation of base shears is the change in the spectrum calculation such that the building does have a higher computed base shear as the period increases.

Under NBCC 2015 to get S(0.2) we now have two equations:

$S(0.2) = F(0.2)S_a(0.2)$ or

$S(0.2) = F(0.5)S_a(0.5)$

The larger of the two equations is to be used.

Under the NBC 2015, both $S_a(0.2)$ and $S_a(0.5)$ are examined for short period buildings as the desire is not to have lower required forces for the design of stiffer buildings than for the design of more flexible buildings. If we have a building that has a fundamental period of say 0.2 seconds, it will soften during a severe seismic event and the period will shift into the higher value (say 0.3 seconds). What this would mean is for a short period building the forces would increase as the event progressed. For taller buildings, their position on the spectrum results in a decrease in forces as the event progresses and the structure softens due to yielding in the plastic hinge zones. For short buildings, NBC 2015 accounts for the softening in the rising spectrum region by requiring us to design with the loads that would be appropriate for a softer structure. See Figure 13.5, which explains this in cartoon form.

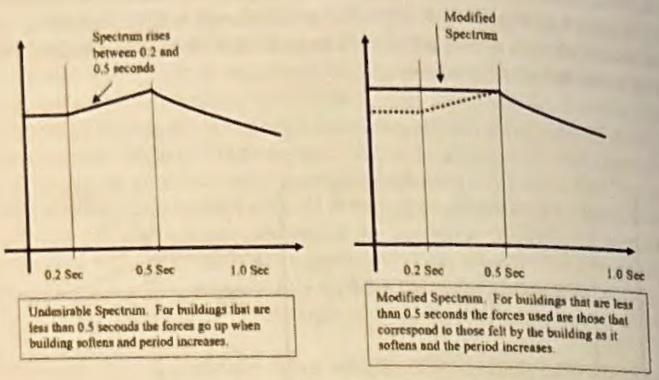
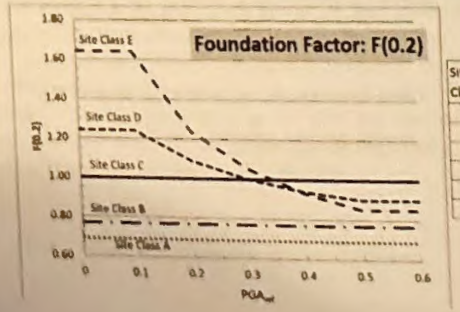


Figure 13.5. Cartoon spectrums showing how the spectrum under NBCC 2015 adjusts to prevent short period buildings from undergoing increasing force as the building softens

The values of $S_a(0.2)$, $S_a(0.5)$ etc. are computed on bedrock of site class C. The effect of the seismic excitation will change if the site is either denser rock than site class C or the site is covered with softer material. This effect is accommodated with the foundation factor, under the NBCC 2015 a new method for computing the foundation factors was introduced. Instead of having foundation factors which applied at all periods there are different foundation factors corresponding to all the periods where $S_a(T)$ is given in Appendix C. From the standpoint of a short period building the most important of these is F(0.2) and F(0.5). The foundation factor is dependent on the period, the site class and a term PGA_{ref} which will be discussed shortly. There is a different foundation factor for all of the key periods (0.2 seconds, 0.5 seconds, 1 second, 2 second, 5 seconds, and 10 seconds). These six foundation factor tables replace the two tables in NBCC 2010 that used to develop F_a and F_v.

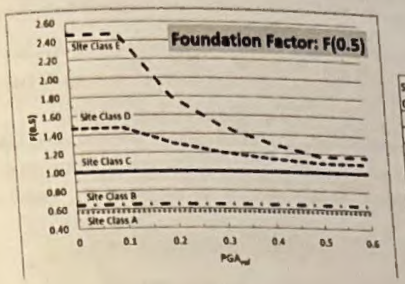
Under NBC 2015 the values of F(0.2), F(0.5), F(1.0), and F(2.0), F(5.0) and F(10.0) are given in table form such as shown in Table 13.1 and Table 13.2. It is useful to represent these values graphically to get a much better impression of how the foundation factor changes for the various site classes and as a function of PGA_{ref}. Using a graphical approach also helps to steer the user away from using an unreasonable number of significant figures as measured values of the foundation factor are subject to considerable scatter. Shown in Table 13.1 are the values for F(0.2) while in Table 13.2 are the values for F(0.5) the graphical representation of both these foundation factors is shown to the left of the table.

Table 13.1. F(0.2) in table form as given in the Code and in graphical form as computed from the tabular values



Site Class	PGA_{ref} <0.1	PGA_{ref} =0.2	PGA_{ref} =0.3	PGA_{ref} =0.4	PGA_{ref} >0.5
	0.1	0.2	0.3	0.4	0.5
A	0.69	0.69	0.69	0.69	0.69
B	0.77	0.77	0.77	0.77	0.77
C	1.00	1.00	1.00	1.00	1.00
D	1.24	1.09	1.00	0.94	0.90
E	1.64	1.24	1.05	0.93	0.85

Table 13.2. F(0.5) in table form as given in the Code and in graphical form as computed from the tabular values



Site Class	PGA_{ref} <0.1	PGA_{ref} =0.2	PGA_{ref} =0.3	PGA_{ref} =0.4	PGA_{ref} >0.5
	0.1	0.2	0.3	0.4	0.5
A	0.57	0.57	0.57	0.57	0.57
B	0.65	0.65	0.65	0.65	0.65
C	1.00	1.00	1.00	1.00	1.00
D	1.47	1.30	1.20	1.14	1.10
E	2.47	1.80	1.48	1.30	1.17

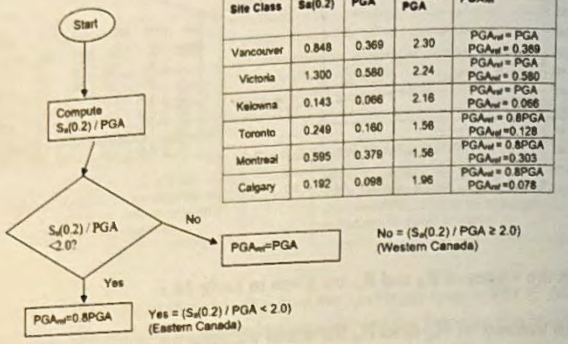
For steel structures the values of R_d and R_o are given in Table 13.3.

Table 13.3. The values of R_d and R_o for steel structures

Type of SFRS	R_d	R_o	R_d*R_o
Ductile moment resisting frames	5.0	1.5	7.5
Moderately ductile moment resisting frames	3.5	1.5	5.3
Limited ductility moment-resisting frames	2.0	1.3	2.6
Moderately ductile concentrically braced frames	3.0	1.3	3.9
Limited ductility concentrically braced frames	2.0	1.3	2.6
Ductile buckling-restrained braced frames	4.0	1.2	4.8
Ductile eccentrically braced frames	4.0	1.5	6.0
Ductile plate walls	5.0	1.6	8.0
Limited ductility plated walls	2.0	1.5	3.0
Conventional construction of moment-resisting frames, braced frames or plate walls	1.5	1.3	2.0
Other steel SFRS(s) not defined above	1.0	1.0	1.0

These can also be found in Table 4.1.8.9 of NBCC which also gives height restrictions for each seismic system depending on the severity of the seismic risk. For example, a moderately ductile concentrically braced frame with tension-compression braces can be a maximum of 60m high in a high seismic risk area while "Other Seismic Force Resisting Systems (SFRS) not defined above" are marked as NP which means that they are NOT PERMITTED for use in high seismic risk areas. Those systems that are marked "NL" have no height limitation.

The determination of PGA_{ref} is dependent on the location of the site. In Western Canada PGA_{ref} is equal to PGA. However, sites in Eastern Canada where seismic events have stronger shaking over larger areas using $PGA_{ref.} = 0.8*PGA$. This can be shown in flowchart and table form in Figure 13.6.



Site Class	Sa(0.2)	PGA	Sa(0.2)/ PGA	PGAref
Vancouver	0.848	0.369	2.30	PGAref = PGA PGAref = 0.369
Victoria	1.300	0.580	2.24	PGAref = PGA PGAref = 0.580
Kelowna	0.143	0.066	2.16	PGAref = PGA PGAref = 0.066
Toronto	0.249	0.160	1.56	PGAref = 0.8PGA PGAref =0.128
Montreal	0.595	0.379	1.56	PGAref = 0.8PGA PGAref =0.303
Calgary	0.192	0.098	1.96	PGAref = 0.8PGA PGAref =0.078

Figure 13.6. Determination of PGA_{ref} the value is different in eastern and western Canada

Under NBCC 2015, the base shear of a building with period less than 0.5 seconds is the larger of the following two equations:

$$V = \frac{2}{3}\frac{S(0.2)I_E}{(R_d R_o)} \text{ And } V = \frac{S(0.5)I_E}{(R_d R_o)}$$

However $S(0.5) = F(0.5) * S_a(0.5)$ and

$S(0.2)$ is the larger of $S(0.2) = F(0.2)*S_a(0.2)$ or $S(0.2) = F(0.5)*S_a(0.5)$

Substituting for $S(0.2)$ and $S(0.5)$ we get the following three equations with the largest of which being the governing base shear:

$$V = \frac{2}{3}\frac{S(0.2)I_E}{(R_d R_o)} \text{ and } V = \frac{2}{3}\frac{S(0.5)I_E}{(R_d R_o)} \text{ and } V = \frac{S(0.5)I_E}{(R_d R_o)}$$

As $V = \frac{2}{3}\frac{S(0.5)I_E}{(R_d R_o)}$ will always be less than $V = \frac{S(0.5)I_E}{(R_d R_o)}$

We reduce to two equations and the base shear of a building having an R_d of 1.5 or greater with a period less than 0.5 seconds and not located on site Class F is the larger of the following two equations:

$$V = \frac{2}{3}\frac{S(0.2)I_E}{(R_d R_o)} \text{ and } V = \frac{S(0.5)I_E}{(R_d R_o)}$$

The static procedure can be best explained through the use of worksheet approach that can be covered in a single page. A blank version of the worksheet is shown in Figure 13.7.

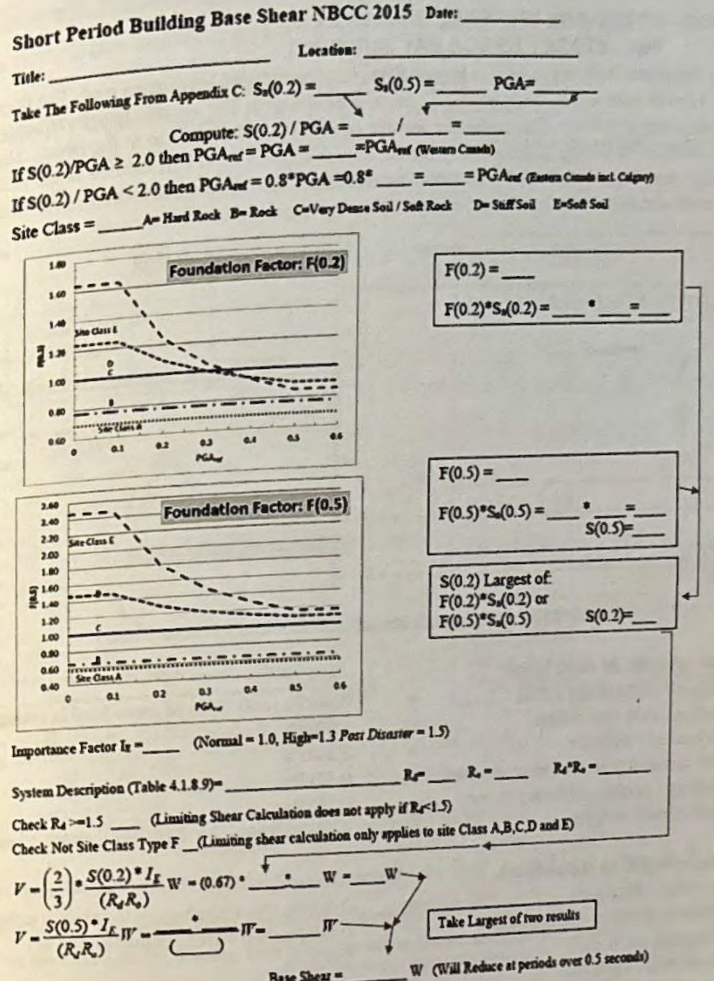


Figure 13.7. Worksheet approach to base shear calculation NBC 2015

The calculation of seismic forces on a two-storey building on a site not subject to liquefaction is best described by an example.

EXAMPLE—DETERMINE THE BASE SHEAR ON A TWO-STOREY BRACE BAY BUILDING

A normal importance building located in Powell River has a suspended level and a roof. The floor-to-floor height is 4.5m on both levels. Each floor is 22.5m x 40m (900m²). The seismic weight tributary to each level is calculated as follows. Determine the seismic base shear and distribute to the floors. The seismic system is moderately ductile concentrically braced frames ($R_d = 3$ $R_o = 1.3$) located at the perimeter of the building as in Figure 13.8. Compute the seismic force on the building; distribute these forces to the various levels and to the braces and then compute the diaphragm design force.

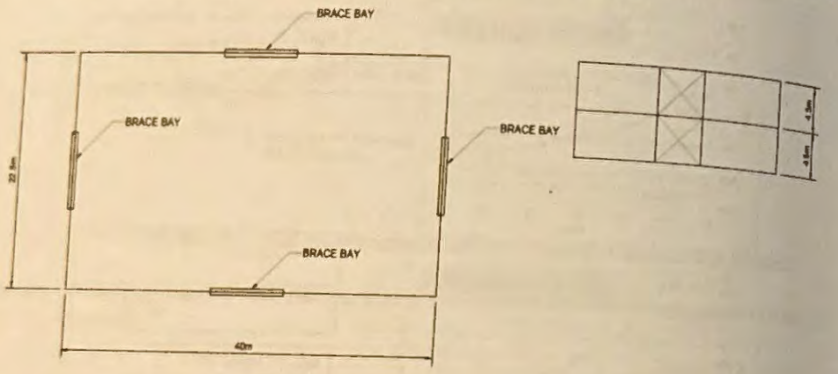


Figure 13.8. Moderately ductile concentrically braced frames

Seismic weight at roof level:

Snow = 0.25*(1.9*0.8 + 0.4)	=	0.48kPa (only 25% of snow load is counted)
Roofing / deck / insulation	=	1.00kPa
Mechanical / electrical	=	0.10kPa
Steel beams & joists & top of column	=	0.32kPa
Cladding / partition tributary to roof	=	0.25kPa
Total seismic weight	=	**2.15kPa (900m² * 2.15kPa = 1935kN)**

Seismic weight at floor level:

Live load (office use)	=	0.00kPa (No contribution to seismic weight)
Partitions & exterior cladding	=	0.50kPa (See NBCC definition for W)
65 topping on 38 deck	=	2.40kPa
Mechanical / electrical	=	0.10kPa
Floor joists	=	0.22kPa
Girders and tributary columns	=	0.10kPa
Total seismic weight at floor level	=	**3.32kPa (900m² * 3.35kPa = 2990kN)**
Total seismic weight = 1935kN + 3015kN	=	**4925kN**

The building is a simple building and if the static design procedure is to provide reasonable results for any structure it would be a simple structure such as this one. We will determine the base shear using the static procedure using a worksheet approach and then determine if the static procedure can be used.

Let us apply the worksheet and compute the base shear of our sample building as shown in Figure 13.9.

Short Period Building Base Shear NBCC 2015 Date: Oct-17-2015

Title: Sample Problem Location: Powell River BC

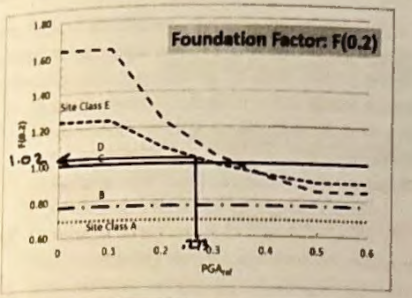
Take The Following From Appendix C: $S_a(0.2)$ = 0.595 $S_a(0.5)$ = 0.556 PGA = 0.273

Compute: S(0.2) / PGA = 0.595 / 0.273 = 2.18

If S(0.2)/PGA ≥ 2.0 then PGA_{ref} = PGA = 0.273 = PGA_{ref} (Western Canada)

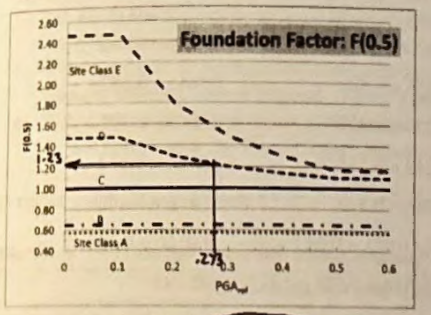
If S(0.2) / PGA < 2.0 then PGA_{ref} = 0.8*PGA = 0.8* ___ = ___ = PGA_{ref} (Eastern Canada incl. Calgary)

Site Class = D A= Hard Rock B= Rock C=Very Dense Soil / Soft Rock D= Stiff Soil E=Soft Soil



F(0.2) = 1.02

F(0.2)*S_a(0.2) = 1.02 * .595 = .61



F(0.5) = 1.23

F(0.5)*S_a(0.5) = 1.23 * .556 = .68

S(0.5) = .68

S(0.2) Largest of:
F(0.2)*S_a(0.2) or
F(0.5)*S_a(0.5) S(0.2) = .68

Importance Factor I_E = 1.0 (Normal = 1.0, High=1.3 *Post Disaster* = 1.5)

System Description (Table 4.1.8.9)= MD Concentric Brace R_d = 3.0 R_o = 1.3 R_d*R_o = 3.9

Check R_d >=1.5 ✓ (Limiting Shear Calculation does not apply if R_d<1.5)

Check Not Site Class Type F ✓ (Limiting shear calculation only applies to site Class A,B,C,D and E)

$$V = \left(\frac{2}{3}\right) * \frac{S(0.2) * I_E}{(R_d R_o)} W = (0.67) * \frac{.68 * 1.0}{3.9} W = 0.12W$$

$$V = \frac{S(0.5) * I_E}{(R_d R_o)} W = \frac{.68 * 1.0}{(3.9)} W = 0.175\ W$$

Take Largest of two results

Base Shear = 0.175 W (Will Reduce at periods over 0.5 seconds)

Figure 13.9. Completed worksheet for seismic base shear calculation

Alternatively if direct numerical interpolation of the foundation factor charts is desired the following would be the calculation of the base shear:

Sample problem. Sunday, September 20, 2015 04:02

Seismic Shear by NBCC 2015 Location: Powell River

$S_a(0.2) = 0.595$ $S_a(0.5) = 0.556$ $S_a(1.0) = 0.373$ $S_a(2.0) = 0.242$ $S_a(5.0) = 0.086$ $S_a(10.0) = 0.031$

PGA = 0.273 PGV = 0.457

Steel-Moderately Ductile Concentrically braced frame $R_d = 3.0$ $R_o = 1.3$ $R_d*R_o = 3.90$

Normal Importance structure $I_E = 1.0$

Site: Stiff Soil Site Class = D

Determine PGA_{ref}: $PGA_{ref} = 0.8PGA$ if $S_a(0.2)/PGA < 2$ but = PGA otherwise

$S_a(0.2)/PGA = 0.595 / 0.273 = 2.18$ $PGA_{ref} = PGA = 0.273$

Use PGA_{ref} to find foundation factors Site Considered to be in Western Canada

Calculate short period spectral acceleration S(T) using NBCC 2015 4.1.8.4(7) provisions:

$S(T) = F(0.2)\ S_a(0.2)$ or $F(0.5)\ S_a(0.5)$ whichever is larger for $T \leq 0.2s$

$F(0.2)*\ S_a(0.2) = 1.02*0.595 = 0.609$

$F(0.5)*\ S_a(0.5) = 1.23*0.556 = 0.682$

For S(0.2): $F(0.5)*\ S_a(0.5) = 0.682$ Governs $S(0.2) = 0.682$ $S(0.2) = 0.682$

T = 0.5sec $S(0.5) = F(0.5)*\ S_a(0.5) = 1.23*0.556 = 0.682$ $S(0.5) = 0.682$

T = 1.0sec $S(1.0) = F(1.0)*\ S_a(1.0) = 1.33*0.373 = 0.497$ $S(1.0) = 0.497$

T = 2.0sec $S(2.0) = F(2.0)*\ S_a(2.0) = 1.38*0.242 = 0.334$ $S(2.0) = 0.334$

T = 5.0sec $S(5.0) = F(5.0)*\ S_a(5.0) = 1.43*0.086 = 0.123$ $S(5.0) = 0.123$

T = 10.0sec $S(10.0) = F(10.0)*\ S_a(10.0) = 1.38*0.031 = 0.043$ $S(10.0) = 0.043$

By interpolation between $S(2.0) = 0.334$ and $S(5.0) = 0.123$ get $S(4.0) = 0.193$

By 4.1.8.4(7) $F_a = F(0.2) = 1.02$ $F_v = F(1.0) = 1.33$ (For use in seismic triggers)

$I_E*F_a*S_a(0.2) = 1.00 * 1.02 * 0.60 = 0.609$ Moderate seismic zone

$S(0.2)/S_a(5.0) = 0.682 / 0.123 = 5.55$

Braced Frames $M_v(0.5) = 1.00$ $M_v(1.0) = 1.00$ $M_v(2.0) = 1.00$

Calculate Base Shear by 4.1.8.11 (Equivalent Static Procedure) at T = 0.225 Seconds

At T = 0.23 Seconds Higher Mode Factor $M_v = 1.0$

$S(T_a) = S(0.225) = 0.682$

By 4.1.8.11(2): $V = S(T_a)*M_v*I_E*W/(R_dR_o) = 0.682*1.00*1.0*W/(3.0*1.3) = 0.175W$

Moment Frame, Braced Frame or Other: Must Exceed: $V = S(2.0)*Mv*I_E*W/(R_dR_o)$

Must Exceed: $V = 0.334*1.000*1.0*W/(3.0*1.3) = 0.086W$

As $R_d >= 1.5$ we need not exceed larger of

$V = 0.67*S(0.2)*I_E*W/(R_d*R_o)$ and $V = S(0.5)*\ I_E\ *\ W\ /\ (R_d\ *\ R_o)$

$V = 2/3\ *\ S(0.2)*I_E/(Rd\ *\ Ro) = 2/3\ *\ 0.682\ *\ 1.0\ /\ (3.0\ *\ 1.3\ *\ W) = 0.117W$

$V = S(0.5)\ *\ I_E/(Rd\ *\ Ro) = 0.682\ *\ 1.0\ /\ (3.0\ *\ 1.3) = 0.175W$

Governing Base Shear = 0.175W

Base shear = 0.175W * 4925kN = 862kN

The "exact" numerical solution of 0.175g confirms the 0.18g found using the graphical selection of foundation factors and the worksheet approach.

13.5 DETERMINING IF STATIC PROCEDURE CAN BE USED

Under NBCC the default seismic design procedure is to perform dynamic analysis; however, we are permitted to use static analysis if the building satisfies **any** of the criteria outlined in Clause 4.1.8.7(1). Paraphrasing and commenting on these clauses we get the following:

a) In cases where $I_EF_aS_a(0.2)$ is less than 0.35. In our case substituting $S_a(0.2) = 0.884$ in the equation for the short spectral acceleration ratio we get: $I_E*F_a*S_a(0.2) = 1.0\ *\ 1.10\ *\ 0.884 = 0.968$ this is considerably greater than 0.35 and of no assistance in high seismic zones such as Vancouver but this clause will be of use in lower seismic zones including some parts of the interior of British Columbia and much of the central Canada.
b) In regular structures less than 60m in height with a fundamental period less than 2.0 seconds. This is probably true but involves reviewing all the irregularity criteria laid out in Table 4.1.8.6.
c) The structure has any type of irregularity except irregularity type 6 (weak storey), 7 (torsionally sensitive) and 9 (Gravity induced lateral demand) and the building is less than 20m in height and has a fundamental period of less than 0.5seconds in each direction.
d) We are required to use time step non-linear dynamic analysis on structures with irregularity type 9 (Gravity Induced Lateral Demand). This is a new type of irregularity and will be discussed shortly.
e) While we are permitted to use static analysis for weak storey structures (irregularity type 6) we are not permitted to have weak storey structures in high seismic zones.

This can be summarized in Table 13.4.

Table 13.4. Permitted uses for static analysis in seismic design

	Low Seismic Risk Area	$I_EF_aS_a(0.2)<0.35$	$I_EF_aS_a(0.2)\geq 0.35$
Regular Building and <60m in Height and T<2 seconds	Static Analysis is expected though Dynamic Analysis can also be used	Static Analysis Permitted	Static Analysis Permitted
Regular Building and ≥60m in Height or T≥2 seconds		**Dynamic Analysis required**	**Dynamic Analysis required**
H<20m and T<0.5Sec. Regular or Irregular but Irregularity NOT: Type 7 (Torsional) Type 9 (Gravity) Type (6 Weak storey)		Static Analysis Permitted	Static Analysis Permitted
Irregular Type 7 (Torsional) Type 9 (Gravity)		**Dynamic Analysis required**	**Dynamic Analysis required**
Type 6 (Weak storey)		Structure Permitted only if $I_EF_aS_a(0.2)<0.2$ and then only with restrictions	**Structure Not Permitted**
Note: Low seismic risk area is defined as $I_EF_aS_a(0.2) < 0.16$ and $I_EF_aS_a(2.0)<0.03$			

With the brace bays on four sides at the edge of the building it can be shown that our structure is not torsionally sensitive and its regular shape can be shown to not have any gravity induced lateral demand so the use of static analysis is appropriate. For many small steel buildings in the range of one to four storeys tall static analysis will be the method used by most engineers to determine the seismic forces on the building.

13.6 DISTRIBUTE THE SEISMIC SHEAR TO THE BUILDING FLOOR LEVELS

The shear is distributed to all levels using the formulas outlined in Building Code Clause 4.1.8.11(6) and as the building period is less than 0.7 seconds there is no shear concentrated at the roof and $F_t = 0$. Force at suspended level 2 (roof level) = F_2 while force at level 1 is F_1.

$$F_x = V * \frac{W_x * h_x}{\sum_i W_i h_i}$$

For the top floor of a two-storey building this reduces to:

$$F_x = V * \frac{W_x * h_x}{\sum_i W_i h_i} = V * \frac{W_2 * H_2}{(W_1 * H_1) + (W_2 * H_2)}$$

$$F_1 = 862kN * \frac{2970kN * 4.5m}{(1900kN * 9m) + (2970kN * 4.5m)} = 378kN$$

$$F_2 = 862kN * \frac{1900kN * 9m}{(1900kN * 9m) + (2970kN * 4.5m)} = 484kN$$

For buildings over two floors, this information can be set out in much more preferable tabular form, as shown in Table 13.5.

Table 13.5. Distribution of seismic sheer over floor levels

Level	Area	Wt / Area	Wi	Hi	Wi Hi	WiHi / Sum(WiHi)	V (kN)	% g
Roof	900	2.15	1935	9	17415	0.564	484	25%
Second	900	3.35	2990	4.5	13455	0.436	378	13%
		Sum =	4925		30870	1.000	862	

In a symmetric building with equal brace bays at the outside edges, a first estimate of the forces on the brace bays can be taken by dividing the shear in each direction on the building by the number of brace bays. This would produce the load on each brace bay as shown in Figure 13.10.

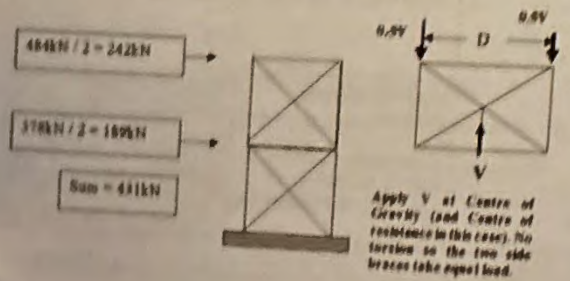


Figure 13.10. Forces on the brace bays assuming no eccentricity

13.7 EFFECT OF TORSION ON THE BRACE FORCES

The effect of torsion must be considered using provisions in the Building Code. A building that is not torsionally sensitive (B<1.7) is symmetric when its centre of gravity coincides with its centre of resistance. We apply the force at each level at an eccentricity of 10% of the building dimension perpendicular to that force. For our building, the upper bound of the torsional effect can be computed by assuming that the braces on the long side of the building resist all the torsion, as shown in Figure 13.11.

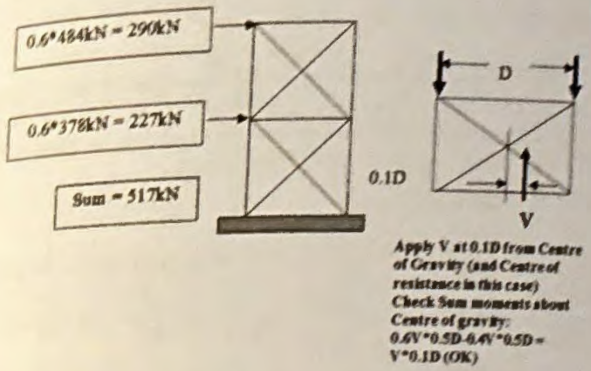


Figure 13.11. Forces on the brace bay including the effect of eccentricity

13.8 DETERMINE THE TORSIONAL SENSITIVITY USING THE COEFFICIENT B

The value of B is a measure of the torsional sensitivity of the building. For a torsionally weak building the value of B will be 1.7 or greater. B is defined in sentence 4.1.8.11(9) and is paraphrased as follows:

$$B = \frac{\text{Maximum storey displacement at extreme point of diaphragm}}{\text{Average storey displacement of the diaphragm}}$$

However, we can save ourselves the effort of computing the deflections as we have a symmetric structure with brace bays at the outside of the building and we don't have issues of amplification of diaphragm displacements beyond the brace bay. By using the principle that in our case the diaphragm deflections are directly related to the forces on the brace bays we get the following:

$$B = \frac{\text{Maximum force in brace bay with torsion considered}}{\text{Force in brace bay with torsion ignored}} = \frac{517kN}{431kN} = 1.2$$

B is less than 1.7 and our assumptions about not having a torsionally sensitive building are valid. We can also show that as the columns are vertical and there are no large cantilevers there is no Gravity Induced Lateral Demand irregularity. We will design the brace bays in the building such that there is no weak storey. Therefore we are permitted to use static analysis to evaluate this building.

Summary of force calculation by static means: The seismic forces have been developed for a simple two-storey steel brace bay building located on site Class D in Vancouver. For this type of structure hand analysis is possible. Much reduction in calculation effort is achieved if it is realized that the governing equation for this type of structure will be the limiting or "cut-off" equation.

13.9 NOTIONAL LOADS AND SEISMIC DESIGN

While the calculation of seismic forces using the procedures given by NBCC applies to buildings of all materials, the use of notional loads is a philosophy unique to steel structures. S16 Clause 8.7.2 requires that we add a translational effect of 0.005 times the factored gravity loads appropriate for that load case. In our case the factored vertical load is equal to the seismic weight. The notional loads are therefore:

Top floor = 0.005 * 1900kN = 9.5kN = 4.8kN / brace

Middle floor = 0.005 * 4870kN = 24.4kN = 12.2kN / brace

In high seismic risk areas these forces tend to get lost. The forces on the braces are already getting increased for accidental eccentricity and will again be increased for overstrength of the brace bay. Even without the increase for overstrength, the notional loads in this case amount to only 3% of the shear at the base of the brace bay when eccentricity is considered as illustrated in Figure 13.12.

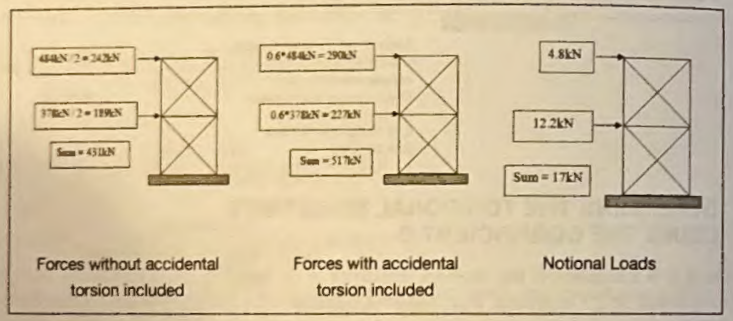


Figure 13.12. Forces on the brace bays including notional loads

We are required to take into account the notional loads when sizing the elements of the seismic force resisting system but once the capacity design procedures are in place it is not required or desirable to add the notional loads to the capacity design forces. The significance of the notional loads on the design is a function of the level of seismic forces. While the notional loads are usually much less than the seismic design loads in high risk seismic zones they are required by S16 to be computed and accounted for in the design of steel buildings in all seismic zones and may become a significant addition to the seismic load in low seismic risk areas.

13.10 TYPE 9 IRREGULARITY – GRAVITY INDUCED LATERAL DEMAND

A new trend in architectural design is to have buildings with large cantilevers or inclined columns. While both of these can make the building look dramatic they both have an undesirable effect on the seismic response of the building. Severe examples of this architectural form are called "Gravity Induced Lateral Demand Irregularity and NBCC 2015 introduced a new irregularity type 9 to make sure that this type of building is analyzed properly. The challenge with gravity induced lateral demand is that the seismic system does not have a symmetric behaviour – the gravity induced demand favours the load in one direction on the seismic system. A ratcheting effect can occur where the structure tries to migrate or wander in one direction increasing the P-Delta effects to the point where collapse can occur. In order to analyze this type of structure it is necessary to do dynamic analysis and not just modal analysis which does not differentiate between one direction or other in a non-symmetric system but it must be non-linear time step analysis where the effect of the seismic softening of the system and the tendency to wander in one direction can be

picked up by the model. Figure 13.13 shows the effect of inclined columns on the gravity induced lateral demand in the lateral system.

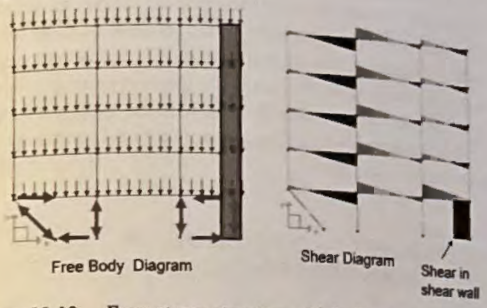


Figure 13.13. Four-storey structure with inclined columns at base leading to gravity induced lateral demand in the seismic system

The Code establishes the severity of gravity induced lateral demand by introducing the term α which is the ratio of the shear due gravity induced lateral demand on the seismic force resisting system to the resistance of the yielding mechanism required to resist the earthquake loads.

$$\alpha = \frac{Q_G}{Q_V}$$

Where Q_G = Gravity induced lateral demand at the critical level of the yielding system
Q_V = Resistance of the yielding mechanism required to resist the minimum earthquake loads but need not be taken less than R_d multiplied by the computed earthquake shear.

The method of finding if there is an issue with the type 9 irregularity is to do the following:

a) Using a computer model or hand analysis with free body diagrams determine the shear in the SFRS due to gravity loads. This should include the appropriate component of live load or snow that should be used with seismic loading.
b) Using a computer model determine the shear on the seismic system at the critical level of the building (often but not always the base of the building). As an initial estimate multiply this shear by R_d.
c) Determine the ratio α.
d) Compare the results to those in Table 13.6. For normal steel systems the seismic force resisting system does not have self-centring characteristics so we are looking at the lower limits of α under "other systems."

Table 13.6. Code limits on α where we are in the gravity induced lateral demand range

Systems with self-centring characteristics	Other Systems	Code Requirement
$0.0 \le \alpha < 0.1$	$0.0 \le \alpha < 0.03$	No requirements
$0.1 \le \alpha < 0.2$	$0.03 \le \alpha < 0.06$	-Multiply displacements by **1.2** -Not allowed in post-disaster buildings
$0.2 \le \alpha$	$0.06 \le \alpha$	Nonlinear response history analysis Not allowed in post-disaster buildings

For most steel buildings the system is not self-centring – traditional steel brace-bay buildings and moment-frame buildings are not considered to be self-centring so the tighter requirements of "other systems" will usually apply. Some innovative systems involving pre-stressing strands have been developed to act as self-centring and some heavily reinforced shear walls are also considered to be self-centring. If the building is found to have a type 9 irregularity then we can try to modify the system such that there is no gravity induced lateral demand; for example, in the case of the four-storey structure shown in Figure 13.13, we can try to convince the architect to straighten the column to the point where it is no longer an issue. Alternatively we can realize that we need to deal with irregularity by computation and that linear static analysis is not the appropriate analysis method.

13.11 TYPE 6 IRREGULARITY – DISCONTINUITY IN CAPACITY (WEAK STOREY)

Weak storey failures of buildings are unfortunately common in seismic events. Weak storey failure often occurs at the base of a building where the shear is highest and the building has be opened up to allow for better circulation. While the most likely location for a weak storey is at the base of the building the failure can occur at other levels in a building. A weak storey is one in which the storey shear strength is less than that in the storey above where the storey shear strength is the total strength of all seismic-resisting elements of the SFRS sharing the storey shear. Weak-storey irregularity is a serious seismic problem as all the inelastic yielding in the building will get concentrated in the weak level. From an analysis standpoint irregularity type 6 causes building behaviour that is not picked up by linear static analysis. For these reasons we are not permitted to have irregularity type 6 in our buildings except in very low seismic risk zones and even then it should be avoided. While static analysis is permitted for irregularity type 6 the building configuration is not permitted. Irregularity type 6 may often occur at the same time as irregularity type 1 which is a vertical stiffness irregularity in which the stiffness (as opposed to the strength) of the seismic force resisting system is less than 70% of that of an adjacent floor.

13.12 LARGE DIAPHRAGM PERIOD SHIFT

The seismic provisions for determining the period of a building under NBCC were for the most part developed with consideration of taller buildings in mind. In typical taller buildings the period decreases ad the height of the building increases. However many buildings in Canada are single-storey buildings with a large footprint and a steel roof diaphragm that has some flexibility. These buildings are usually designed with no intermediate braces which would interrupt internal activities and the span of the diaphragm is much greater than the height of the brace bays at the sides of the building. Typical examples of such buildings are shown in Figure 13.14.The construction type is typical of many single-storey warehouse buildings, including big-box retail stores.



Figure 13.14. Typical large footprint single-storey buildings that can be seen when flying near any airport in Canada. Many of these buildings are located on soft sites near rivers

Ambient vibration studies have shown that the period of these buildings is much more dependent on the length of the longest dimension of the diaphragm than on the height of the braces at the outside of the building. To accommodate these findings NBCC 2015 allows the period to be shifted on single-storey buildings with large diaphragms to take account of the increase in period that would occur. If the period shift is sufficiently large then a lower base shear corresponding to the higher period can help to reduce the forces on the building including the diaphragm.

The period that we are allowed to use is:

a) $0.05\ (h_n)^{3/4} + 0.004\ L$ for shear walls,

b) $0.035\ h_n + 0.004\ L$ for steel moment-frames and steel braced-frames

Where hn is the height of the single-storey brace or wall at the diaphragm ends and L is the diaphragm clear-span between brace bays or end walls.

EXAMPLE—DETERMINE THE PERIOD OF THE BUILDING

A large single-storey building is located in site class E in Richmond BC and has moderately ductile concentrically braced bays only on the exterior of the building. In the long direction the distance between brace bays is 100m and the height of the brace bay is 8m. Determine the period of the building and show that the base shear forces decrease.

Solution:

The period is calculated using new formulas from NBCC 2015 that consider the large diaphragm with brace bay formula which is appropriate for this building:

$T = 0.035\ h_n + 0.004\ L$ for steel moment frames and steel braced frames

$T = 0.035*8m + 0.004*100m = 0.68sec$

This is illustrated in Figure 13.15.

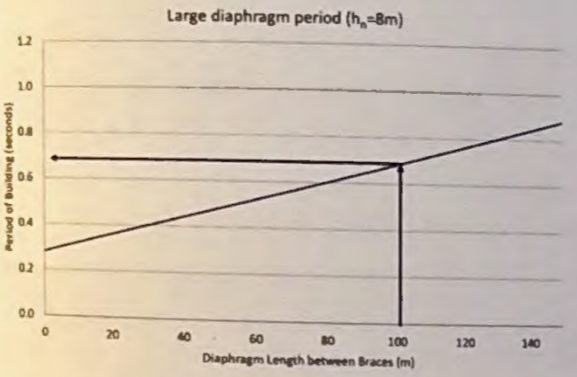


Figure 13.15. Period shift for a building with 8m high brace bays and a diaphragm span of 100m between brace bays

The formula for the period of the steel brace bay building if it did not have a flexible diaphragm is: $T = 0.025h_n^{(3/4)}$

Putting in our brace height of 8m we get: $T = 0.025*h_n^{(3/4)} = 0.025*(8m)^{0.75} = 0.12$ sec

We can then plot the two periods on the design spectrum for the site which includes the system ductility factors and site class factors as shown in Figure 13.16.

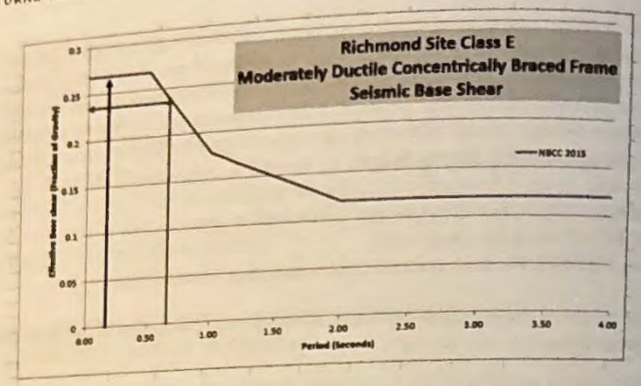


Figure 13.16. Design spectrum for a moderately ductile concentrically braced frame located on site class E in Richmond. The periods of 0.12 seconds (dashed arrow) and 0.68 seconds (solid arrow) have been shown

Figure 13.16 shows the effect that the period shift has on the base shear. Using the brace bay only period 0.12 seconds we get 0.27g while with the diaphragm modified period of 0.68 seconds we get 0.23g. The degree of the reduction in forces will depend on the site location and site class with more dramatic reductions where the soil under the foundation is more solid than site class E.

There are practical limits to how long the building can go between brace bays. Thermal movement and loads will get too high if the building is too long. Buildings that exceed 100m will often have expansion joints to help control this and it would be normal to have a brace bay each side of the expansion joint.

We have determined the forces on the system under seismic loads and we are now in a position to look at capacity design and the lateral force resisting system.

CHAPTER 14
CAPACITY DESIGN AND THE LATERAL FORCE RESISTING SYSTEM

This chapter continues our discussion of seismic systems by focusing on seismic force resisting systems (SFRS) and concentrically braced bay design. We open with a general discussion of the brace bays or other force resisting systems that are necessary to take the forces from the diaphragms to the foundation. We then move on to reviewing what constitutes a concentrically braced seismic resisting frame and how to design these systems.

14.1 THE LATERAL FORCE RESISTING SYSTEM

In Chapter 13 we looked at the forces that occur on the roof and floor diaphragms of a building. We also looked at how to design the diaphragm to transfer the forces to the shear-resisting elements that carry these forces to the ground. In this chapter we will examine brace bays as one type of shear-resisting element, but before doing so, it is important to review where these elements are located. Ideally each diaphragm will have a shear- resisting element on four sides as shown in Figure 14.1.

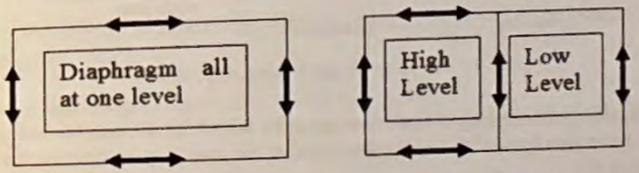


Figure 14.1. Shear-resisting elements of roof and floor diaphragms

Shear-resisting elements are often concealed in walls or, if the walls are constructed of concrete or masonry, the walls themselves will act as shear-resisting elements.

It may not be possible to put brace bays at the edge of the diaphragm for all cases; for example, if it is desired to keep one of the walls open or if there is a covered loading dock at one end of the building, as shown in Figure 14.2.

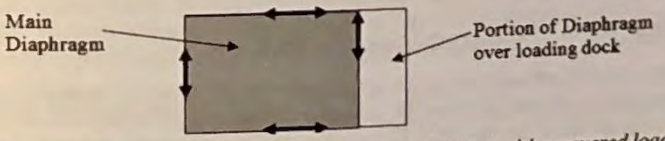


Figure 14.2. Location of shear-resisting elements in a building with a covered loading dock

In these cases, the diaphragm must cantilever past the last shear-resisting elements. This situation requires a rational consideration of the forces involved, including design of the diaphragm chords to account for the cantilever. Design of the last brace bay for the amplified forces of the cantilever will also be necessary.

This situation can be taken to the extreme when designing what is often referred to as a three-sided building; for example, a retail store with a large glass storefront, such as are used in brand-name shops on major shopping streets in most cities. Here we will often have very large shear-resisting elements on the sides of the building caused by concrete or masonry firewalls and a short wall at the rear of the building. Also at the rear of the building will be found any elevator or stair core that could assist in resisting shear, leaving the front area available for high-end retail. This can result in high diaphragm shears at the rear wall and high drifts at the front of the building, especially if the diaphragm flexibility is accounted for in the model. The solution adopted by most designers is to place a moment frame at the open front of the building as shown in Figure 14.3.

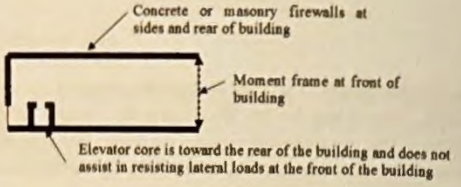


Figure 14.3. Moment frame at open front of three-sided building

Another problem arises if the diaphragm becomes too slender, which may result in excessive diaphragm deflection and excessive diaphragm shears. The solution here is to add interior shear-resisting elements to make each portion of the diaphragm have an aspect ratio (long side / short side) closer to 2 or less, as shown in Figure 14.4.



Figure 14.4. Interior shear-resisting elements

In Figure 14.4 not only have shear-resisting elements been added in the interior but additional shear-resisting elements have also been added to the long side of the building so that the shear forces do not have to be dragged so far.

The location of interior brace bays must be decided in conjunction with the architect and will require communication between both parties. Typically we start with locating the interior brace bays in interior architectural partition walls; however, while these locations will often be acceptable at design time, they may pose challenges while renovating the building in the future. The location of brace bays in plumbing walls is to be avoided, if possible, as the braces inevitably conflict with exhaust vents and plumbing lines. Brace bays in exterior walls will be less problematic during future renovations but must be coordinated with the location of windows and doors.

14.2 OVERSTRENGTH AND CAPACITY DESIGN

As an example of overstrength and capacity design, consider that a frame is to be built to support a weight. You are told the factored weight (W_F) may be at least 65kN but can go higher. You are going to support the weight on a rod so that, if the weight is too great, it will just stretch the rod and allow the weight to rest on the surface below as shown in Figure 14.5a.

You must find the maximum load on the rod and make sure that the frame has sufficient capacity to support the rod. You start by trying to find a rod that has the assumed force-displacement properties shown in Figure 14.5b.

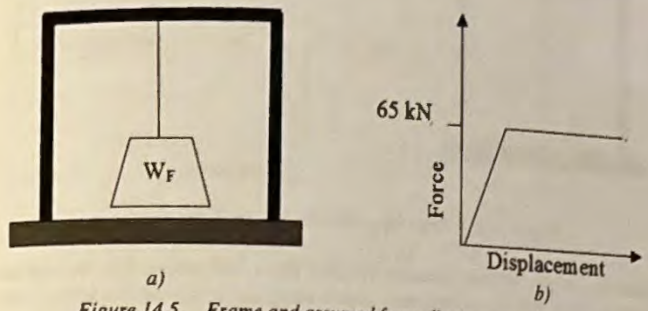


Figure 14.5. Frame and assumed force-displacement properties

For 65kN we need an area of the rod of:

$$A_{req} = \frac{65kN}{(\phi = 0.9) * (F_y = 300000)} * 10^6 = 240mm^2$$

The available rod sizes are as shown in Table 14.1.

Table 14.1. Available rod sizes

Rod size	Area (mm^2)
½-inch diameter	127
5/8-inch diameter	198
3/4-inch diameter	285

The 5/8-inch rod is too small to support the minimum factored load so we will select the 3/4-inch diameter rod with an area of 285mm^2.

The **Factored Resistance** of the rod is $T_r = \emptyset * Area * F_y$

$T_r = 0.9 * 285mm^2 * 300{,}000kN/m^2 * 10^{-6}$

$T_r = 76.95kN$

But the rod won't yield at that point. Instead the rod yields when loaded to the yield stress and we must remove the Ø factor to get the nominal resistance.

The **Nominal Resistance** of the rod is $T_n = Area * F_y$

$T_n = 285mm^2 * 300{,}000kN/m^2 * 10^{-6}$

$T_n = 85.5kN$

But then we must consider that the actual yield of the steel in the rod will be higher than the specified yield. The Code says that, when doing capacity design, the 300MPa steel should be considered to be 385MPa. Applying this true yield strength we get the probable capacity.

The **Probable Tension Capacity** of the rod is $T_u = Area * (R_y F_y)$

$T_u = 285mm^2 * 385{,}000kN/m^2 * 10^{-6}$

$T_u = 110kN$

We must therefore design our frame using capacity design of 110kN as shown in Figure 14.6.

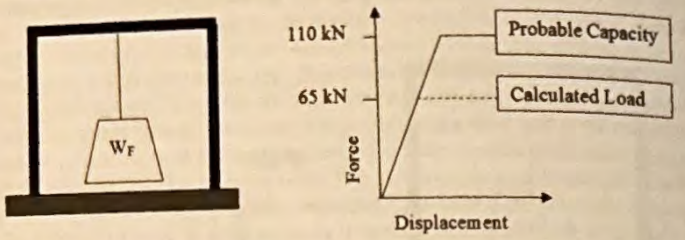


Figure 14.6. Probable capacity and calculated load

In this case, capacity design load from the rod at 110kN is about 70% greater than the original "must be greater than" load of 65kN. This is very similar to seismic design in that we start out with a Code-specified minimum force level that we design to and then find the probable capacity of the yielding element (in this case the rod); we then make sure that all the remaining elements in the load path (in this case the frame) are designed for that capacity design load. For efficient capacity design, we must try to get the probable capacity as close to the calculated load as we can to minimize the loads that we have to design all the parts of the structure that are not yielding.

14.3 PERMITTED SEISMIC SYSTEMS FOR STEEL BUILDINGS

The Building Code has several categories and associated R_dR_o-values for seismic systems used in steel buildings. While some steel buildings rely for seismic support on concrete walls, tiltup or masonry, seismic systems can also be constructed purely from steel. The basic steel seismic resisting systems are:

1) Braced systems, including concentrically braced, eccentrically braced, and chevron braced, tension-compression braced system or tension-only braced systems and buckling restrained braces
2) Moment frames
3) Steel plate shear walls
4) Conventional construction which has a lateral capacity but not the ductile detailing that ensures significant excursions beyond the yield capacity of the system, including cantilever column systems where the vertical columns are the only hinging element

These systems, along with their R_dR_o values and the Code clauses that provide the minimum criteria that must be met for each system, are shown in the flowchart in Figure 14.7.

The main systems of braced bay, moment-frame, and steel plate shear walls have two and sometimes three R_dR_o values, depending on the level of ductility that can be provided through the detailing.

Two points must be kept in mind when selecting seismic systems for steel buildings:

1) Building Code Clause 4.1.8.9(3) requires that, if there are two seismic systems acting in the same direction, then the minimum R_dR_o value for the seismic systems on that direction must be used for computing the force on all seismic resisting elements in that direction. An $R_d = 1.5$ masonry wall for the south wall of the building and an $R_d = 5$ moment frame on the north wall requires that the steel moment frame must be designed using $R_d = 1.5$ forces. When mixing systems in one direction, care should be taken to make certain the systems are compatible and have similar stiffness properties, as a very flexible system on one side of the building and a very rigid system on the other side will lead to twisting of the building.
2) It is permitted to use different R_dR_o values in the east-west direction than those in the north-south direction, but it is not advised to mix systems that have very different stiffness properties. For example, a flexible moment frame in one direction would not mix well with a rigid masonry wall system in the other direction.

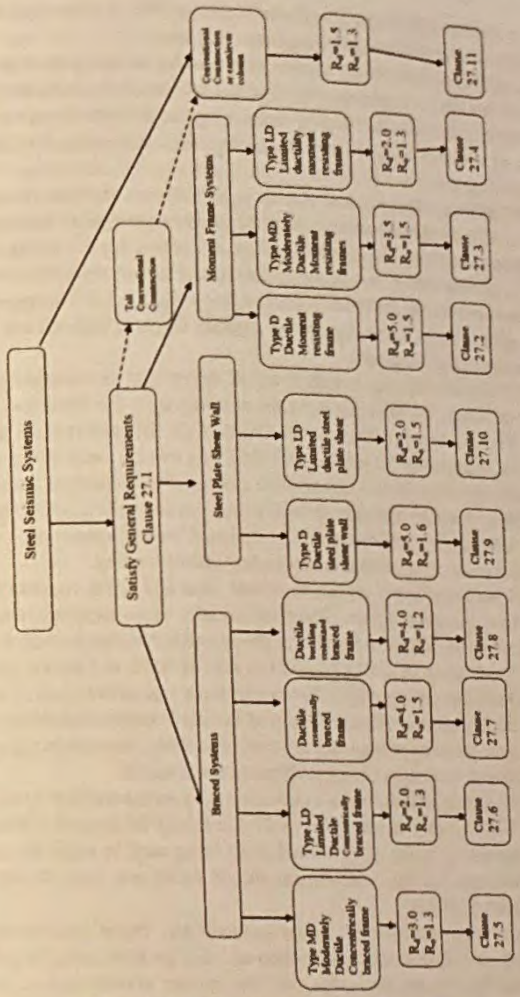


Figure 14.7. Flowchart of steel seismic systems

14.4 GENERAL REQUIREMENTS OF CLAUSE 27.1

Provided a seismic system with an R_d-value greater than 1.5 has been chosen, the seismic system must meet the general clauses of Clause 27.1 as well as the clauses that apply specifically to the seismic system chosen. There are now nine general requirements in Clause 27.1 and it is important to go through these on a point-by-point basis, as follows:

1) Clause 27.1.1 tells us that we have to apply the requirements of Clause 27.1 and the appropriate system requirements for systems with R_d greater than 1.5. Seismic systems that have an R_d of 1.5 or less need only comply with Clause 27.10. Clause 27.1.1 also tells us that we have to design

our seismic system with an eye to both the Building Code requirements and the steel standard requirements. Building Code requirements would include:

- The method of distributing the force to the elements, including the distribution of force up the structure and the calculation of effects of real and accidental eccentricity.
- The inter-storey drift calculated by multiplying the deflections calculated by $(R_dR_o) / I_E$ is to be a maximum $0.01h_s$ for post-disaster buildings and $0.02h_s$ for schools and $0.025h_s$ for other buildings

2) Clause 27.1.2 tells us we have to use capacity design. It is necessary to choose a yielding element and make sure that all elements connected to this element can deliver the forces necessary to yield it. This applies to diaphragms, columns footings and beams that are acting as collector elements. Clause 27.1.2 also includes a very important line telling us that, regardless of the result of capacity design, we need not exceed the forces corresponding to $R_dR_o = 1.3$. This provision comes in handy in lightly loaded systems where developing capacity forces is difficult and in systems that have poorly defined yielding locations.
3) Clause 27.1.3 says that we have to fully detail all the parts of the seismic system and show these parts on our drawings. We can't just leave the detailing up to the fabricator.
4) Clause 27.1.4 tells us to make sure that those parts of the structure that are not part of the seismic system must be designed to "go along for the ride" in a seismic event without failure. An example of a situation to avoid would be a non-seismic moment frame that fails under imposed deflection or P-Delta effects when the structure deflects from seismic loading. Splices in gravity columns in multi-storey buildings are now required to be designed for a minimum shear that takes account of the deformation of the member that occurs under seismic loading.
5) Clause 27.1.5 has restrictions on the structural steel and weld material that can be used in the seismic force resisting system. There are actually three requirements in this clause. The requirements are more restrictive for thicker pieces and for higher seismic zones, as follows:
   - 27.1.5.1: Steel has to be G40.20/G40.21 or ASTM A992 and have a specified yield of 350 MPa or less except gravity only columns may have a specified yield of 450 MPa.
   - 27.1.5.2: For higher seismic zones (most of British Columbia would be considered a higher seismic zone) thicker sections that are part of seismic force resisting systems or "welded parts" shall have minimum Charpy V-Notch impact values.
   - 27.1.5.3 For higher seismic zones welds of primary members shall have filler material with a Charpy V-Notch impact test value of 27 J at minus 30 degrees Celsius. This requires a considerable change in the types of weld sticks being used by many fabricators with definite cost implications and this requirement should be shown very clearly on the structural drawings prior to tender.
6) Clause 27.1.6 has requirements on bolted connections. These requirements include using pretensioned bolts in bolted seismic connections and they prohibit using long slotted holes. Short slotted holes must be perpendicular to the load. The number of bolts is sized using factored loads and bearing capacity of the bolts but the bolts must still be pretensioned.
7) Clause 27.1.7 introduces the factor R_y which is a factor to account for the probable overstrength of the members when doing capacity design. When looking at overstrength of the seismic system we will usually look at the product R_yF_y.
   - For 350 MPa steel (Except HSS) R_y is 1.1 giving R_yF_y of 385 MPa
   - For 300 MPa steel, the Code recognizes that much 300 grade material is actually stronger than 300MPa and requires the R_yF_y product to be 385MPa for 300 MPa steel in angles and plates.
   - For HSS members the product of R_yF_y has increased in S16-09 to 460 MPa which increases R_y to 1.37 this reflects the cold working of the plates produces higher strength steel that does not yield at the levels expected which is unconservative when sizing the adjacent members. R_y is computed by back-calculation from the minimum R_yF_y of 460 MPa (R_y = 460 MPa / 350 MPa = 1.37). This will increase the strength requirements for columns of the brace bay and will increase diaphragm forces on buildings using HSS brace members. (Previously the requirements under S16 R_yF_y for HSS members were 385 MPa.)

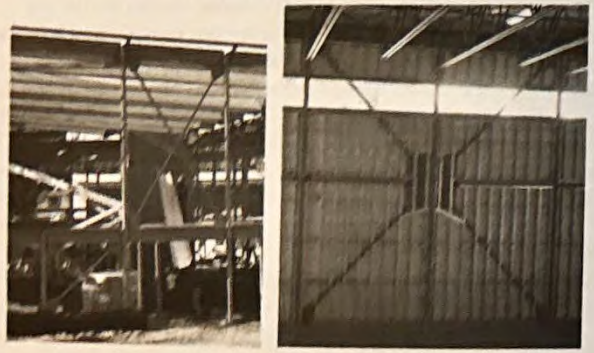
8) Clause 27.1.8 requires the U_2 factor used when determining P-Delta effects to be increased by R_d to account for the actual drift of the structure under seismic loading. This is mostly an issue with moment frames and cantilever structures. Notional loads are not required to be considered when the yielding forces of members are placed on the structure.
9) Clause 27.1.9 introduces the concept of "protected zones." The aim is to provide protection to the zones where the inelastic action will take place. In the protected zone, the fasteners, welds and holes are not permitted, including those for attaching mechanical, electrical or architectural features. The exception is that decks may be spot welded in the protected zone but may not be pinned or screwed. Each of the systems in Clause 27 lists the extent of the protected zone.

CHAPTER 15
SEISMIC DESIGN OF CONCENTRICALLY BRACED FRAMES

Having satisfied the general requirements of Clause 27.1, we will now look at the system of moderately ductile concentrically braced frames. We will examine brace bays on a clause-by-clause basis to see what is required to be satisfied to make the system work, including examining some of the practical detailing issues. We will examine the moderately ductile concentrically braced frame ($R_d = 3$). As we will see under S16, almost all of the moderately ductile concentrically braced requirements are required to be satisfied when designing the lower ductile ($R_d = 2$) braced bays, and there may be few advantages to going to the lower R_d-value.

15.1 CONCENTRICALLY BRACED STEEL FRAMES

Concentrically braced steel brace frames or "brace bays" are used in British Columbia to provide lateral stability to low-rise buildings for wind and earthquake forces. Brace bays are frequently found in one- to three-storey buildings and may be used in higher buildings to provide lateral support for mechanical penthouses located on the roof. Figure 15.1 shows two typical brace bays.



a) *b)*

Figure 15.1. Typical brace bays

Ideally, the brace bay should possess the following properties:

a) Resistance to in-plane lateral forces due to wind and seismic loading with good behaviour under factored loading
b) Easy to design as the braces are often used on small buildings where extensive analysis is unwarranted and unfunded

c) Slender enough to fit into a stud wall which in the case of most schools and low-rise buildings may use 152mm studs
d) Economical to fabricate and install
e) Good behaviour under service conditions

Frequently the shear that brace bays carry is quite low. A minimum number of brace bays will usually be required in a building to provide both lateral resistance in each direction and suitable torsional resistance, as well as to limit diaphragm stresses.

With the advent in 1994 of the requirement to connect for the tension capacity of the brace, there was a move in the design community to reduce the tension capacity of the brace so that the forces on columns and footings could be reduced. It is obviously not cost effective to design the connections, connected members and foundations to resist several times the imposed load.

15.2 ASSUMED BEHAVIOUR OF BRACE

The brace is assumed to carry only axial loads. Braces with KL/r ratios of 200 or less are assumed to have both tension and compression capacities. To determine the factored resistance of the brace, it is assumed to have a force displacement curve as shown in Figure 15.2. The brace does not suddenly lose capacity when the buckling capacity is reached, but rather is assumed to continue to initially carry a force corresponding to the connection design force in compression, which then gradually reduces to the post-buckling capacity.

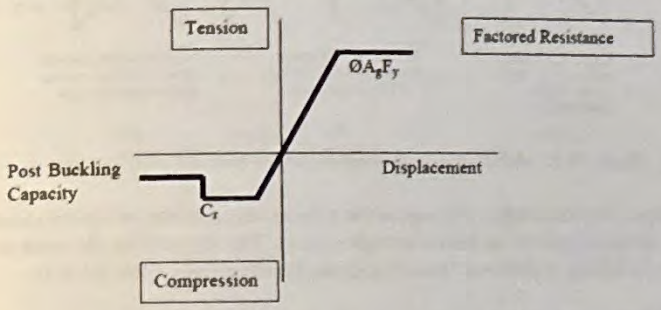


Figure 15.2. Idealized force displacement curve of typical brace element

While Figure 15.2 shows the assumed behaviour for factored resistance, the actual behaviour will be different and we must account for overstrengths in the yielding element.

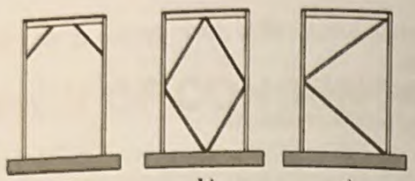
15.3 BRACE CONFIGURATION

Clause 27.5.2.1 tells us that the concentrically braced system applies to:

a) Tension-compression bracing systems
b) Chevron bracing systems
c) Tension-only bracing systems
d) Other systems that are similar and have stable inelastic response

The frames that are NOT permitted are as follows:

a) Knee braced frames.
b) K braces where the braces meet at a column between floors. Figure 15.3 illustrate some brace bays that would not be permitted. (The shaded rectangle at the top of the brace indicates a roof or floor deck – there is no floor or roof level at mid-level for these figures.)



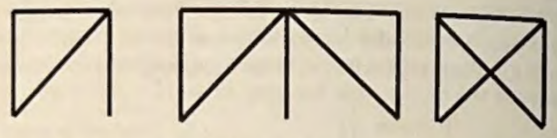
a) *b)* *c)*

Figure 15.3. Brace configurations that are NOT permitted

In addition, Clause 27.5.2.2 requires the braces be proportioned such that approximately half the horizontal force on any plane frame is taken in tension elements in each direction. This is expressed more or less as follows:

$$0.75 \le \frac{Tension\_Brace\_Capacity\_in + X\_direction}{Tension\_Brace\_Capacity\_in - X\_direction} \le 1.33$$

Following this requirement (which applies to $R_d = 3.0$ and $R_d = 2.0$ Concentrically braced systems as it is not modified in Clause 27.6) braced systems in which there are braces in only one direction are not acceptable, as shown in Figure 15.4.



a) *b)* *c)*

Figure 15.4. Additional brace configurations that are NOT permitted

The other restriction on configuration is to require that tension-only systems be limited to four storeys and that tension-compression systems be limited to eight storeys. This depends on the *short period spectral acceleration* ($I_E F_a S_a(0.2)$) as shown in Table 15.1 (from Building Code Table 4.1.8.9).

Table 15.1. Limiting heights for various braced systems

TYPE OF SRFS	MAXIMUM HEIGHT FOR CASES WHERE $I_E F_A S_A(0.2)$			
	<0.2	≥2.0 to <0.35	≥0.35 to ≤0.75	>0.75
Moderately Ductile concentrically braced frames (Rd=3.0 Ro=1.3)				
Tension-Compression braces	NL	NL	40m	40m
Chevron braces	NL	NL	40m	40m
Tension only braces	NL	NL	20m	20m
Limited Ductility concentrically braced frames (Rd=2.0 Ro=1.3)				
Tension-Compression braces	NL	NL	60m	60m
Chevron braces	NL	NL	60m	60m
Tension only braces	NL	NL	40m	40m

Source: Building Code Table 4.1.8.9

In Table 15.1, the abbreviation "NL" means "no limit"; in other words, the system is not limited in height in low seismic zones. In contrast, all of southwestern British Columbia would fit into the right-hand

column regardless of the site class, where the short period spectral acceleration is influenced by soil conditions at the site.

15.4 FORCES ON A SAMPLE BRACE BAY

To cover the principles in designing a single-storey brace frame, we will determine the expected forces on a brace bay in a sample single-storey building. We will then take those forces and look at several solutions for handling them.

EXAMPLE—CALCULATE FORCES ON A SAMPLE BRACE BAY

A normal importance single-storey building is to be located on very dense soil (Site Class C) in an area where the seismic risk values and site class give a base shear of 0.11g under $R_d = 3.0$, $R_o = 1.3$. The building is rectangular in plan with dimensions 30mx50m and has a moderately ductile concentrically braced bay ($R_d = 3.0$, $R_o = 1.3$) at each of the exterior walls as shown in Figure 15.5. The seismic weight is computed as follows:

Snow = 0.25*(2.4*0.8 + 0.3)	=	0.56kPa
Roofing / deck / insulation	=	0.80kPa
Mechanical / electrical	=	0.20kPa
Steel beams & joists & top of columns	=	0.30kPa
Cladding / partition tributary to roof	=	0.25kPa
Total seismic weight	=	**2.11kPa**

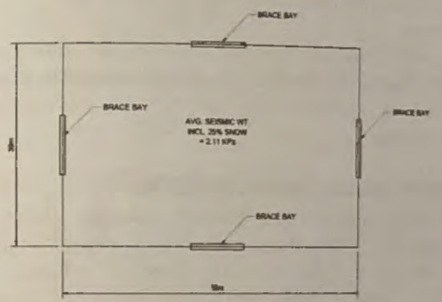


Figure 15.5. Rectangular building with concentrically braced bay

Solution:

The base shear is computed using the provisions of Chapter 13 and found to be 0.11g.

Base Shear = **0.11 W** (will reduce at periods over 0.5 seconds)

Forces at end brace bays including a 20% increase to account for eccentricity effects are:

$V_{brace\ bay}$ = (50m/2) *(30m)* 2.11kPa *0.11 * 1.2 = 208kN

Notional loads of 0.005 * 2.11kPa * (50m/2) * (30m) = 7.9kN should then be added to the brace for sizing the members in the brace. Once the members in the brace are sized, the notional loads are not added to the capacity design brace forces.

15.5 BRACE BAY DESIGN SEQUENCE

We now have the calculated seismic load on the brace bay. The next issue is to design it. Use the following sequence:

1) Compute the forces on the brace bay including notional loads.
2) Produce a brace bay configuration that meets code requirements.
3) Place the computed forces on the brace bay.
4) Select members that have sufficient capacity to resist the computed forces.
5) Compute the connection design forces in the brace. These are the capacity design loads, and for tension members, are computed as $R_y A_g F_y$.
6) Compute the "bailout" loads corresponding to $R_d R_o = 1.3$ forces.
7) Design columns and other elements, including the diaphragm for the lesser of the probable capacity design forces or the $R_d R_o = 1.3$ loads.

The configuration of a tension-compression brace must meet the following criteria:

- Braces to intersect columns only at floor levels.
- In high seismic zones braces to not exceed 20m in height if tension only or 40m if tension-compression.
- No Knee or K bracing.
- Braces have tension elements taking 75% to 133% of load regardless of direction of load.
- Braces have members with a maximum slenderness ratio (KL/r) of 200 but can be made up of:
  - HSS members (round, square, or rectangular)
  - Rolled sections including angles, channels, and wide flanges
  - Built-up sections
- In single- and two-storey buildings brace bays have columns without splices, while in buildings with more than two storeys, they must have continuous columns with splices at least two storeys apart.
- Use Class 1 or 2 columns.

EXAMPLE—DETERMINE BRACE CONFIGURATION

Continuing the example presented earlier, a single-storey concentric tension-compression braced bay of height of 3.5m and width of 7.0m with two inclined concentric braces is carrying a factored seismic shear of 208kN is to be designed and detailed to satisfy Clause 27.5 (Moderately Ductile Concentrically Braced Frames $R_d = 3.0$.) The proposed scheme (as shown in Figure 15.6) determines whether or not the configuration meets requirements.

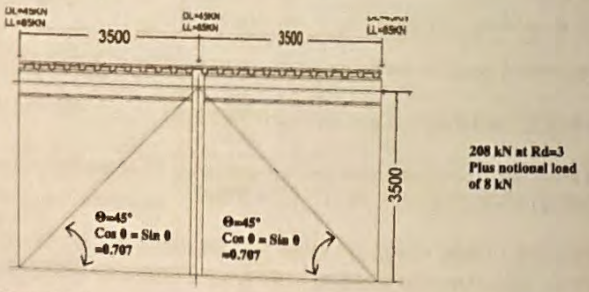


Figure 15.6. Brace-configuration scheme for compression brace bay

Solution:

The brace configuration meets Code requirements and satisfies the requirements set out in Table 15.2 (shown as OK in checklist format). ($L_{brace} = 4.94$m)

Table 15.2. Code requirements for brace configuration

CLAUSE	REQUIREMENT	OK?
27.5.2.1	No knee bracing or K bracing - pairs of braces or single braces do not meet one side of column between floors.	OK
27.5.2.2	Tension braces in both directions with capacity 0.75 to 1.33 of compression braces	OK
27.5.2.3	For moderate seismic and above tension compression systems <40m building height increase forces over 32m	OK
27.5.2.4	This clause applies to chevron bracing and can be ignored for the type of concentric bracing we are using here where braces are at beam / column intersections	OK
27.5.2.5(a)	Tension only systems: <=20m in height - increase forces if over 16m high	OK
27.5.2.5(b)	Tension only systems: Fully continuous columns	OK
27.5.2.5(c)	Tension only systems: Column splice restrictions for buildings with greater than 2 storeys.	OK
27.5.3.1	KL/r for brace min of 200	
27.5.3.2	For KL/r <=100: Rect. and sq. HSS b/t<17.6 Round HSS b/t<28.6	
27.5.3.2	For KL/r <=200: HSS to be Class 1	
27.5.4.1	Minimize eccentricity in connecting elements	

Other items on the checklist cannot be verified yet. They will have to be satisfied as our design progresses.

The structure is to be designed as tension-compression and is less than 40m high.

15.6 SIZING THE BRACE MEMBERS

Our first task in the design of the brace bay is to distribute the forces to the individual elements. As the structure is symmetric, the force in the left brace will be the same as the right brace and can be computed by dividing the horizontal load applied to the brace by two and then converting the force to the angle of the brace, as follows:

Total load on the two braces is 208kN + 8kN notional load

Before buckling of the compression brace each brace takes a horizontal load equal to half this.

[(208kN + 8kN) / 2)] / 0.707) = 153kN

The 0.707 is the cosine of the angle that each brace makes with the horizontal which in our case is 45 degrees, as shown in Figure 15.7.

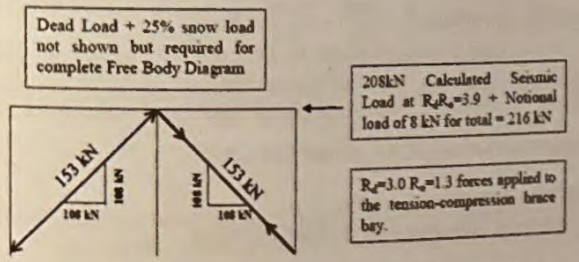


Figure 15.7. Free body diagram showing brace angle

The unbraced length of the brace element is 4.95m and we can determine several possible members of this length that will resist a compression of 153kN. Several options are listed in Table 15.3. The compression capacity C_r is taken from a linear interpolation of the column capacity tables (green pages) of the *CISC Handbook*. G40.21 Class C sections have been used.

Table 15.3. Size of brace members

Brace	Area (mm^2)	R (mm)	KL/r	C_r (kN)	T_r (kN)	T_r/C_r
HSS 89x89x 9.5	2790	31.7	156	181	879	4.9
HSS 102x102x4.8	1790	39.2	126	167	564	3.4
HSS 114x114x3.2	1390	45.1	110	160	438	2.7
HSS 127x127x4.8	2280	49.6	100	302	718	2.3
HSS 114Øx4.8	1640	38.8	128	150	517	3.4

We will initially choose the member with the minimum area to reduce the connection forces and then check to make sure the size chosen meets the KL/r and b/t requirements.

Before going any further we shall check to make sure that the brace meets the requirements of the Code with respect to KL/r. Clause 27.5.3.1 requires that the maximum value of KL/r for the brace is to be 200. All the braces selected above satisfy this requirement (this check is easy to make – if the brace has a compression value in the column capacity tables (green pages) of the *CISC Handbook* at the KL/r being evaluated then it is at least less than 200).

We must now check that this member size is compatible with the required b/t ratios.

15.7 REQUIRED B-OVER-T RATIOS FOR SQUARE AND RECTANGULAR HSS SECTIONS (CLAUSE 27.5.3.2)

Under Clause 27.5.3.2 there are limits on b-over-t (b/t) ratios depending on the KL/r ratio. This allows for elastic buckling without the premature failure due to wall necking that can occur if the b/t ratio is too high. Clause 11.3.2(b) allows us to compute the b/t ratios for square and rectangular HSS sections using the nominal outside dimension less four times the wall thickness.

Under Clause 27.5.3.2 the limits of b/t for square and rectangular HSS members used as diagonal braces in moderately ductile concentrically braced frames are as follows:

For square and rectangular HSS, KL/r <= 100 use $\frac{b}{t} = \frac{330}{\sqrt{Fy}} = \frac{330}{\sqrt{350}} = 17.6$

For square and rectangular HSS, KL/r = 200 use Class 1 = $\frac{b}{t} = \frac{420}{\sqrt{Fy}} = \frac{420}{\sqrt{350}} = 22.45$

Use linear interpolation between KL/r = 100 and KL/r = 200.

For round HSS, KL/r <= 100 use $\frac{b}{t} = \frac{10000}{Fy} = \frac{10000}{350} = 28.6$

For round HSS, KL/r = 200 use Class 1 = $\frac{b}{t} = \frac{13000}{Fy} = \frac{13000}{350} = 37.1$

Use linear interpolation between KL/r = 100 and KL/r = 200

EXAMPLE—DETERMINE BRACE SIZES

Determine if our preliminary brace sizes have acceptable b/t ratios

Solution:

The initially chosen member of HSS 114x114x3.2 has a KL/r ratio of:

KL/r = 1.0 * 4940mm / 45.1mm = 110 **(OK < 200)**

HSS 114x114x3.2 has a b/t ratio as follows:

$b/t = \frac{114 - (4 * 3.18)}{3.18} = 31.8$ This is over the requirement of 17.6 for KL/r of 17.6 and 22.45 for members with a KL/r of 200 and is therefore **not acceptable** for our KL/r of 110.

Try HSS 102x102x4.8 braces as it can be shown that they have an acceptable b/t ratio for the tension-compression bracing system and a low cross-sectional area that will be a benefit during overstrength calculations. (We will stay away from round HSS 114Øx4.8 due to the difficulty in getting round sections in G40.21 and the uncertainty in the wall thickness and lower specified yield of 317 MPa for the ASTM sections.) Select an HSS 102x102x4.8 for the brace member.

Check HSS 102x102 x 4.8 HSS for use in the brace bay:

KL/r = 1.0 * 4940mm / 39.2mm = 126 **(OK < 200)**

HSS 102x102x4.8 has a b/t ratio as follows:

$b/t = \frac{102 - 4 * 4.8}{4.8} = 17.25$

Required for KL/r = 100 max b/t = 17.6 for KL/r = 200 max b/t = 22.45

For KL/r = 126 by linear interpolation max b/t = 17.6 + (126 – 100)(22.45 – 17.6)/100 = 18.5

OK – our b/t of 17.25 is less than 18.5 **Section is acceptable for use**

We now have selected a brace size that meets force and b/t ratio requirements. The initial design force on the brace bay will now have little importance to us now as we are doing capacity design and will substitute the forces on the brace bay will now result from the yielding of this member not from the calculated loads from our calculated seismic distribution.

15.8 DETERMINING THE BRACE CAPACITY FOR TENSION-COMPRESSION LOADS

As there are a limited number of sizes of member that can be selected the brace will inevitably have more capacity than we need. We may wish to determine the capacity of the braced frame at the point where the compression brace buckles.

EXAMPLE—DETERMINE THE MAXIMUM HORIZONTAL LOAD

Continue previous example using the 102x102 x 4.8 brace and determine the maximum horizontal load at $R_d = 3$ $R_o = 1.3$ load that can occur for the selected brace configuration to be considered Tension-Compression.

Solution:

At a 4.95m unsupported length the compression capacity of the HSS 102x102x4.8 brace is about 169kN (exceeds our seismic load combined with accidental eccentricity and notional load of 153kN). Up to this

point both the tension and compression brace will take equal load and the horizontal load on the brace bay when the compression brace is loaded to its axial resistance will be:

2 braces * 169kN * 0.707 = 238kN

This is about 110% to our design load of 216kN and shows the tension-compression brace is being reasonably efficiently designed.

Figure 15.8 shows a free body diagram of brace bay at the point where factored compression capacity of compression brace occurs.

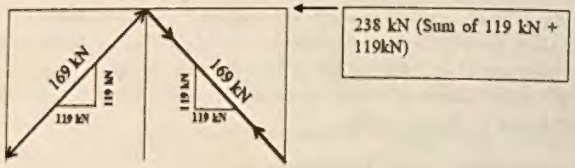


Figure 15.8. Free body diagram of brace bay at the point where factored compression capacity of compression brace occurs

15.9 DETERMINING THE PROBABLE AXIAL CAPACITY OF THE BRACE

Once the brace members have been selected it is time to determine the brace connection forces that will govern all the remaining design of the brace bay. The brace member will have what is termed its "Probable Tension Resistance" and its "Probable Compression Resistance" and "Post Buckling Capacity." These resistances must be determined and placed on the brace bay so that the brace behaviour and overstrength can be evaluated.

S16 very eloquently describes these axial states in the brace that need to be evaluated.

> **27.5.3.4 Probable brace resistances**
>
> For the purpose of evaluating forces on connections and other members upon yielding and buckling of the bracing members in capacity design, the probable tensile resistance of bracing members, T_u, shall be taken as equal to $A_g R_y F_y$; the probable compressive resistance of bracing members, C_u, shall be taken as equal to the lesser of $A_g R_y F_y$ and $1.2C_r/\Phi$, where C_r is computed using $R_y F_y$; and the probable post-buckling compressive resistance of bracing members, C'_u, shall be taken as equal to the lesser of $0.2AgR_yF_y$ and C_r/Φ, where C_r is computer using R_yF_y.
>
> Each of the two loading conditions,
> a) the compression acting braces attaining their probable compressive resistance, C_u; and
> b) the compression acting braces attaining their probable buckled resistance, C'_u,
>
> shall be considered as occurring in conjunction with the tension acting braces developing their probable tensile resistance, T_u.
>
> For chevron bracing, when plastic hinging in the beam is permitted by Clause 27.5.2.4(c) or 27.6.2.2, the brace tensile force need not exceed the greater of that corresponding to plastic hinging in the beam and that corresponding to C_u of the compression brace.
>
> When the forces corresponding to R_uR_v=1.3 are computed, the redistribution of forces due to brace buckling shall be considered.

These forces are computed without using a Ø factor and by increasing the yield by a factor R_y to account for the expected overstrength in the yield of the steel. The probable capacity of the brace members are the loads to be used in designing the connections to the brace and the forces on adjacent elements such as columns and collector beams.

The values of R_y (as laid out in S16 Clause 27.1.7) are as follows:

For G40.21 Steel of F_y = 300MPa R_yF_y = 385 MPa (R_y = 1.28)

For G40.21 steel of F_y = 350 MPa R_yF_y = 385 MPa (R_y = 1.1)

For G40.21 steel with yield above 350 MPa R_y = 1.1

For HSS Sections R_yF_y = 460 MPa (R_y = 1.31)

The special requirements and higher R_y for HSS members reflects research showing that HSS members, possibly due to cold working, will usually have an actual yield strength that is well in excess of the specified yield stress. The desire is that the yielding element protect the remainder of the structure from overloading but if the actual yield is higher than expected then the yielding will occur at a higher value, this is what the R_y factor is protecting against.

15.10 PROBABLE TENSILE RESISTANCE (T_u)

The probable tensile resistance of the brace is determined by removing the resistance factor (Ø) and increasing the yield strength of the material to R_yF_y and is expressed in formula form as:

$$T_u = (R_yF_y)*A_g$$

EXAMPLE—COMPUTE PROBABLE TENSION RESISTANCE

Compute the probable tension resistance of our HSS 114Øx4.8 brace.

Solution:

The probable tension resistance of the member is $(R_yF_y)*A_g$

$$T_u = (R_yF_y)*A_g = 460\text{MPa} * 1640\text{mm}^2 \times 10^{-3} = 754\text{kN}$$

15.11 PROBABLE COMPRESSIVE RESISTANCE (C_u)

The probable compression capacity (C_u) is more complex as the compression capacity will depend on the KL/r ratio and the capacity is not linear as illustrated in Figure 15.9.

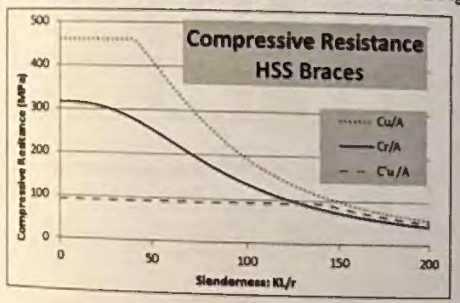


Figure 15.9. Probable compressive resistance of HSS members

To compute the probable compression resistance of a brace we need to use R_yF_y in the compression formulas of Clause 13 and remove the Ø factor producing the following results:

$$\lambda = \frac{KL}{r}\sqrt{\frac{R_yF_y}{\pi^2 E}} \text{ and } C_r = \phi * A * F_y\left(1+\lambda^{(2*n)}\right)^{-\frac{1}{n}}$$

EXAMPLE—COMPUTE THE PROBABLE COMPRESSION CAPACITY OF AN HSS BRACE

Compute the probable compression capacity of our HSS 102x102x4.8 brace. The unsupported length is 4.95m.

Solution:

$$\lambda = \frac{KL}{r}\sqrt{\frac{R_y F_y}{\pi^2 E}} = \frac{1.0 * 4.95m}{0.0392m}\sqrt{\frac{460MPa}{\pi^2 * 200000MPa}} = 1.927 \text{ (KL/r = 127.3)}$$

$$C_u = 1.2 * A * R_y F_y \left(1 + \lambda^{(2*n)}\right)^{-\frac{1}{n}}$$

$C_u = 1.2 * 1790 \times 10^{-6}m^2 * 460{,}000kN/m^2 * (1 + 1.927^{2.68})^{\frac{1}{1.34}}$

$C_u = 1.2 * 823kN * (6.800)^{-0.746} = 987kN * (0.239) = 236kN$

It would also be possible to use the data in Table 15.4 to determine C_u as follows:

KL/r = 127.3 use KL/r = 128

$C_u/A = 129MPa$ $C_u = (1790 \times 10^{-6}m^2) * (129{,}000kN/m^2) = 231kN$

Table 15.4. Brace capacities for G40.21 HSS

HSS Brace Capacities for G40.21 HSS Fy=350 MPa (Does not apply to ASTM)

KL/r	C_r/A	C_u/A	C'_u/A	KL/r	C_r/A	C_u/A	C'_u/A	KL/r	C_r/A	C_u/A	C'_u/A
0	315	460	92	52	249	400	92	104	127	191	92
2	315	460	92	54	243	389	92	106	123	176	92
4	315	460	92	56	238	379	92	108	120	171	92
6	315	460	92	58	233	368	92	110	117	166	92
8	314	460	92	60	227	357	92	112	114	161	92
10	314	460	92	62	222	347	92	114	111	157	92
12	313	460	92	64	217	337	92	116	108	152	92
14	312	460	92	66	211	327	92	118	105	148	92
16	311	460	92	68	206	317	92	120	102	144	92
18	310	460	92	70	201	307	92	122	100	140	92
20	308	460	92	72	196	298	92	124	97	136	92
22	307	460	92	74	191	288	92	126	95	132	92
24	304	460	92	76	186	279	92	128	92	129	92
26	302	460	92	78	181	271	92	130	90	126	92
28	299	460	92	80	176	262	92	132	88	122	92
30	296	460	92	82	171	254	92	134	86	119	92
32	293	460	92	84	166	246	92	136	84	116	92
34	290	460	92	86	162	239	92	138	82	113	92
36	286	460	92	88	158	231	92	140	80	110	92
38	282	460	92	90	153	224	92	142	78	108	92
40	278	460	92	92	149	217	92	144	76	105	92
42	273	453	92	94	145	211	92	146	74	102	92
44	269	443	92	96	141	204	92	148	72	100	92
46	264	432	92	98	137	198	92	150	71	97	92
48	259	422	92	100	134	192	92	152	69	95	92
50	254	411	92	102	130	187	92	154	68	93	92

KL/r	C_r/A	C_u/A	C'_u/A	KL/r	C_r/A	C_u/A	C'_u/A
156	66	91	91	192	45	62	62
158	65	89	89	194	45	60	60
160	63	87	87	196	44	59	59
162	62	85	85	198	43	58	58
164	61	83	83	200	42	57	57
166	59	81	81				
168	58	79	79				
170	57	78	78				
172	56	76	76				
174	54	74	74				
176	53	73	73				
178	52	71	71				
180	51	70	70				
182	50	68	68				
184	49	67	67				
186	48	66	66				
188	47	64	64				
190	46	63	63				

Condition	Symbol	Calculate
Factored Tension Resistance	T_r	$\varnothing A_g F_y$
Factored Compression Resistance	C_r	Use green pages
Probable tension resistance	T_u	$A_g R_y F_y$
Probable compression resistance	C_u	Lesser of: $1.2*C_r/\varnothing$ computed with $R_y F_y$ or $A_g R_y F_y$
Probable Post Buckling Compression Resistance	C'_u	Lesser of: $C_r/\varnothing$ computed with $R_y F_y$ or $0.2A_g R_y F_y$

15.12 POST-BUCKING COMPRESSION CAPACITY OF THE BRACE (C'_u)

After the brace has gone through a few cycles, the compressive resistance will drop to what is referred to as the post buckling compression capacity. For slender braces, the brace has a post buckling compression capacity that is less than the connection design force for the brace in compression. The post buckling resistance of the member is given in Clause 27.5.3.4 as being the lesser of $0.2A_gR_yF_y$ and $C_r/\varnothing$ where C_r is computed using R_yF_y.

Graphically we can compare the probable compressive resistance, C_u, and the probable post buckling compressive resistance C'_u and see that the probable compressive resistance is considerably greater than the post buckling compressive resistance until we get to kL/r values of 150 or more when the post buckling and the compressive resistance are somewhat similar though not exactly equal.

Continuing our example of the HSS 114x114x4.8 with an unsupported length of 4.95m, we get the probable post buckling compressive resistance as being the lesser of:

$0.2A_gR_yF_y = 0.2*(A_g = 1790 \times 10^{-6}m^2)*(R_yF_y = 460{,}000kN/m^2) = 165kN$

Or

$169kN*(R_yF_y/F_y)/\varnothing = 169kN * (460/350)/0.9 = 246kN$

Using the lesser of two numbers above we get the governing post buckling compression capacity = 165 kN

It would also be possible to use the data in Table 15.4 to determine C'_u as follows:

KL/r = 127.3 use KL/r = 128

$C'_u/A = 92$ MPa $C_u = (1790 \times 10^{-6}m^2)*(92{,}000kN/m^2) = 165kN$

A summary of forces to be considered in our brace is shown in Table 15.5.

Table 15.5. Forces to be considered for HSS braces

Condition	Symbol	Calculate	For our brace
Factored Tension Resistance	T_r	$\varnothing A_g F_y$	564 kN
Factored Compression Resistance	C_r	Use green pages	169 kN
Probable tension resistance	T_u	$A_g R_y F_y$	823 kN
Probable compression resistance	C_u	Lesser of: $1.2*C_r/\varnothing$ computed with R_yF_y or $A_gR_yF_y$	236 kN
Probable Post Buckling Compression Resistance	C'_u	Lesser of: $C_r/\varnothing$ computed with R_yF_y or $0.2A_gR_yF_y$	165 kN

We can then use these forces to replace the brace in capacity design procedures in Figure 15.10 and Figure 15.11.

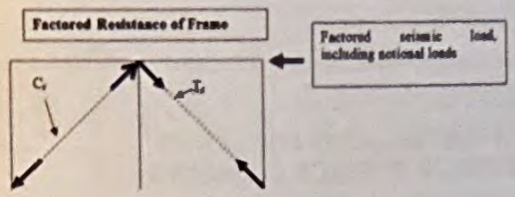


Figure 15.10. Factored resistance of frame

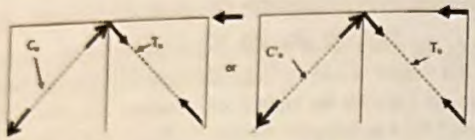


Figure 15.11. Capacity design of frame

When computing overstrength or effects on adjacent elements use the C_u or C'_u whichever produces the worst effect on the element you are considering. If doing a computer model, replace the compression element with the forces and delete the compression brace element from the model.

15.13 OVERSTRENGTH CAPACITY OF THE BRACE BAY

An important number to compute is the overstrength capacity of the brace bay. This will give an indication of its ability to dissipate forces in a ductile manner and will determine what the forces are on the diaphragm and other elements such as the columns.

EXAMPLE—DETERMINE OVERSTRENGTH CAPACITY

Determine the overstrength capacity of the sample brace bay and determine the effective R_dR_o value of the forces that correspond to this.

Solution:

Place the probable resistance forces on the brace bay and determine the resulting horizontal force produced.

Probable Tension Capacity = T_u = 823kN

Horizontal component of tension brace = 0.707 * 823kN = 582kN

Probable Compression Capacity = C_u = 236kN

Horizontal component of Compression brace = 0.707*236kN = 166kN

Total horizontal component = 166kN + 582kN = 748kN

Overstrength capacity of brace bay = 748kN

Overstrength ratio = 748kN / 208kN = 3.6 (ignore notional load when using capacity design forces).

The resulting applied notional forces are shown in Figure 15.12.

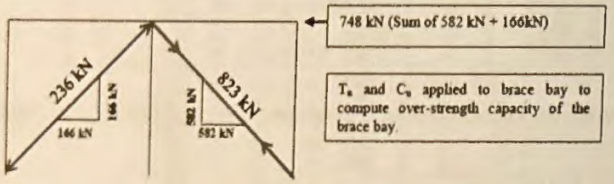


Figure 15.12. Applied notional forces

Effective R_dR_o for forces at overstrength = (R_dR_o = 3.9) * (208kN / 748kN) = 1.08

15.14 CHECKING THE "BAILOUT" FORCES TO LIMIT THE FORCE IN BRACE ELEMENTS

Clause 27.5.4.2 requires that we connect the brace members for their probable tension capacity (T_u) and their probable compression capacity (C_u). However, now we are allowed to limit the tension-connection

forces to those that would exist when R_dR_o = 1.3 forces are applied to the entire building and redistributed due to buckling of the brace.

EXAMPLE—DETERMINE BAILOUT FORCES

For the example brace bay, determine the bailout forces.

Solution:

The first step in determining the forces in the brace elements at the limiting force of R_dR_o = 1.3 is to determine the force that is applied to the frame. We then distribute this force realizing that compression braces cannot carry more load than their compression resistance C_r.

Compute the horizontal force in brace bay at R_dR_o = 1.3

$$\text{Horizontal Force} = \frac{R_dR_o = 3.9}{R_dR_o = 1.3}(208kN) = 624kN$$

When doing the distribution at R_dR_o = 1.3 we are also required (by S16 Clause 27.1.8.1) to consider the notional load. This gives us a force of 624kN + 8kN = 632kN. It is not appropriate to multiply the forces in the brace elements by 3 as the majority of the increase in the load on the frame in going from R_dR_o = 3.9 to R_dR_o = 1.3 forces will be carried by the tension brace. To determine the distribution on our frame, we will put the compressive resistance value C_u on the compression brace and determine how much tension is in the tension brace. The compression brace horizontal component is 0.707*(C_u = 236kN) = 166kN. At R_dR_o = 1.3 horizontal shear of 647kN the horizontal force on the tension brace must now go to 632kN – 166kN = 466kN giving a force in the brace of (466kN/0.707 = 659kN). The R_dR_o = 1.3 forces will cause the tension brace force to go up to 659kN (ignoring yielding) from the 147kN when only R_dR_o = 3.9 forces were applied or 153kN when notional loads plus R_dR_o = 3.9 forces were applied (as shown in Figure 15.13). The tension brace increases by a factor of 4.4 while the force applied to the brace bay goes up by a factor of 3. The 0.707 in the preceding discussion is the cosine of angle that the brace makes with the horizontal and will change for braces that are not at 45 degrees to the horizontal.

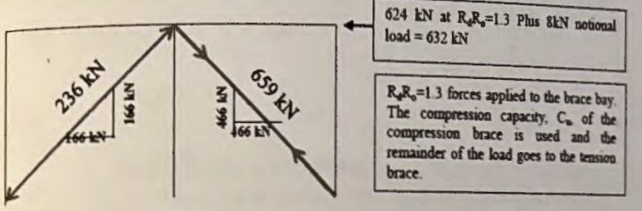


Figure 15.13. Compression brace values

Alternatively we can consider the situation with the compression brace carrying its post buckling capacity of 165kN. The horizontal component of this is 0.707 * 165kN = 117kN which leaves 632kN – 117kN = 515kN as the horizontal component in the tension brace. The tension brace is therefore carrying 515kN / 0.707 = 728kN, as shown in Figure 15.14.

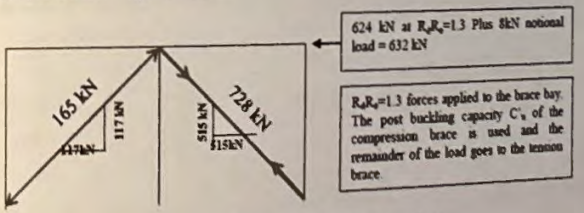


Figure 15.14. Tension brace values

What we see from Figure 15.13 and Figure 15.14 is that, in this case when the $R_dR_o = 1.3$ forces plus the notional loads are applied to the structure, the maximum load in the tension brace is 728kN. This is less than the T_u value of 823kN and we could within S16 and NBCC provisions reduce the connection design force on the brace to 728kN. The aim of this exercise is to illustrate capacity design procedures and this bailout will not be used for the remainder of the example.

15.15 ACCOUNTING FOR THE CONNECTION FORCES ON THE ADJACENT MEMBERS

The Code and engineering logic require that we take account of the brace forces in designing other members in the brace bay. The brace force will influence both adjacent beam forces and column forces, as well as connected foundations. Adjacent beams will have axial forces from seismic loads that must be dragged to the brace bay while adjacent columns will have vertical loads transferred from the braces. Figure 15.15 shows the compression elements that must be examined when the force in the tension brace is developed. The forces in flanges from bending and the unbraced length of the bottom flange complicate design of the beams for compression. A conservative approach to this problem is to do an axial-moment interaction for the beam, assuming that the beam is unsupported for axial force but has its compression flange restrained for moment. In addition to designing these members to carry the axial loads that result from the action of the tension brace it is necessary to include these forces on brace bay elevations shown on the structural drawings so that the steel detailer designing the connection of the beams will address the axial load effect.

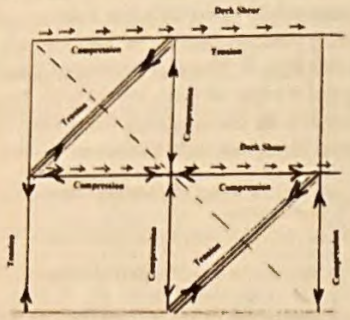


Figure 15.15. compression elements to be examined when designing a brace bay

EXAMPLE—DESIGN CONNECTION FORCES ON A BRACE BAY

Determine the forces in the centre column of our brace bays and design it.

Solution:

We have already shown that the bailout $R_dR_o = 1.3$ forces are less than the capacity design forces on the brace; however, for this illustration we will continue to design for the capacity design forces. The compression in the column can be computed by adding the appropriate dead and live load applied to the top of the column to the vertical component of the tension brace and then subtracting the vertical component of the compression brace at its minimum load which is the post buckling compression capacity C'_u. This would be as shown in Figure 15.16.

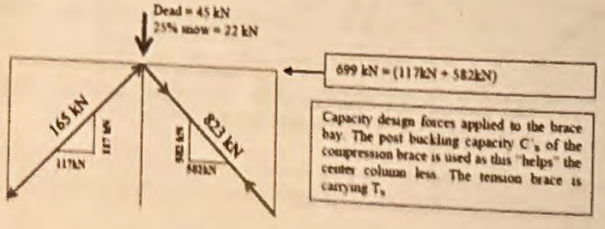


Figure 15.16. Capacity design forces

The resulting loads on the column are as follows:

Dead load = 45kN	= 45kN
25% snow load = 86kN /4 =	= 22kN
Tension brace force = 823kN*0.707	= 582kN
Post Buckling brace force = –165kN*0.707	= –117kN
	= 531kN

It is worth noting how much greater this force is than the force in the column without considering the yielding of the compression brace. In that case, the vertical component of the compression brace balances the vertical component of the tension brace and the only load would be the 45kN dead load and 22kN from the snow load.

While the 531kN is a large axial load we must also consider some moment with this axial load and Clause 27.5.5.2 requires that columns in braced bays shall be Class 1 or 2 and shall have a bending resistance of not less than $0.2ZF_y$ in combination with the computed axial loads. The moment is applied in the plane of the brace bay.

Trial size: $0.2ZF_y$ is approximately 25% of capacity when the resistance factor, Ø, is considered. We will look in the column capacity tables (green pages) of the *CISC Handbook* for a column with an axial capacity of 531kN / 0.75 = 708kN.

We find: an HSS 127x127x 9.5 has a $C_r = 786$kN at $L_u = 3.6$m

$0.2ZF_y = 0.2 * 183 \times 10^3 mm^3 \times 10^{-9} * 350{,}000 kN/m^2 = 12.8kNm$

(Effectively we are applying an eccentricity of 12.8kNm / 531kN = 24mm)

$\kappa = \frac{M_{small}}{M_{large}} = 0$ $\omega_1 = 0.6 - 0.4\kappa \geq 0.4$ as Kappa = 0 use 0.6 for Omega.

For HSS 127x127x 9.5KL/r = 3500 / 48.0 = 72.9 (Table 4-7) $C_e/A = 370$MPa

$C_e = 370{,}000 * 4240 mm^2 \times 10^{-6} = 1569$kN

$$U_1 = \left[\frac{\omega_1}{1 - \frac{C_f}{C_e}}\right] U_1 = \left[\frac{0.6}{1 - \frac{531}{1569}}\right] = 0.90 \text{ <1 use 1}$$

$$\frac{C_f}{C_r} + \frac{U_1 * M_f}{M_r} = \frac{531kN}{786kN} + \frac{1.0 * 12.8kNm}{57.6kNm} = 0.67 + 0\,22 = 0.90 \text{ (OK)}$$

An HSS 127x127x9.5 column meets moment and axial requirements.

The selected brace member sizes ($R_dR_o = 3.9$ tension-compression brace) are shown in Figure 15.17.

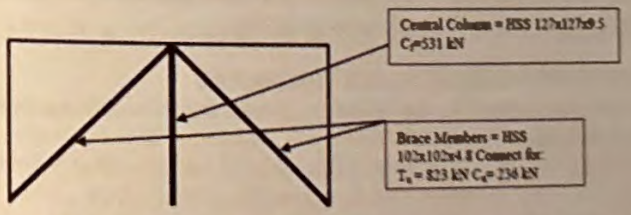


Figure 15.17. Selected brace member sizes ($R_dR_o = 3.9$ tension-compression brace)

15.16 PROVIDING DUCTILE ROTATIONAL BEHAVIOUR IN THE BRACE CONNECTION (27.5.4.3)

The brace connection must be designed to provide ductile rotational behaviour at the ends of the brace members so that, when the elastic buckling of the compression brace occurs, it will not attempt to tear the gusset plate or the brace at its ends. There are two types of connection for the brace element: (1) those that provide a single gusset plate and permit rotation in the gusset and (2) those that provide gussets each side of the brace, which do not permit rotation in the gussets forcing the rotation to occur in the brace element itself. Recent studies have indicated that a more ductile situation can be achieved if the brace is allowed to rotate in the gusset area rather than forcing the rotation into the brace member. The Code in Clause 27.5.4.3 addresses these issues separately with the following requirements:

1) If the connection can permit rotation (single central gusset plate) then provide an end distance of two times the thickness of the gusset plate as illustrated in Figure 15.18.
2) If the connection is such that rotation must occur at the brace ends (side gusset plates) then the moment capacity of the connection plates must be increased to make certain that there are not two hinges forming, one at the brace end and one in the connection. In this case the moment capacity of the connections is handled by capacity design forcing them to have a moment capacity exceeding $1.1*Z_{brace}*(R_yF_y)$.



Figure 15.18. Providing for ductile rotational behaviour

15.17 DETAILING THE BRACE AT THE GUSSET PLATE

The requirement that we connect brace elements for $(R_yF_y)*A_g$ can result in some difficulties when going through the usual requirements for connection of tension elements. Typically the brace area must be reduced at the connection for bolt holes or for slotting for gusset plate welds. An example of the connection of a tension brace for a force equal to 40% of A_gF_y is given in the *CISC Handbook* in Part 5. Substitution of a force equal to $1.1A_gF_y$ as the connection force results in a very large bolted connection. Recall also that all bolted connections for seismic frames must use pretensioned bolts and it is apparent why welded connections are preferred at this location. A welded gusset is often used to connect brace members and this can be sized using the axial forces in the brace.

EXAMPLE—DETERMINE GUSSET PLATE THICKNESS

Determine the gusset plate thickness for our HSS 102x102x4.8 brace. Tension-connection force = T_u = 823kN and compression force = C_u = 236kN.

Solution:

The force from the brace will spread into the gusset plate as we move from first contact point. If we assume conservatively that at the end of the brace a width of gusset plate equal to twice the brace diameter we can compute the thicknesses required.

Tension: $t = 823kN / [(2*0.114m)*(0.9*300,000kN/m^2)] = 0.0149m$ use 15.8mm (5/8-inch)

Compression:

For a plate: $r = \sqrt{\frac{I}{A}} = \sqrt{\frac{\frac{bt^3}{12}}{b*t}} = \sqrt{\frac{t^2}{12}} = \frac{t}{\sqrt{12}}$

For a 15.8mm plate, r = 4.58mm

If we take the unbraced length as 2* 0.0158m = 0.032m and use a buckling k of 2.0 we get kL = 64mm. If we conservatively increase this to 150mm as we can't really determine where the ends of the gusset plate "column" are we get kL/r = 150mm / 4.58mm = 33 and C_r / A for 300MPa gusset material is found from the C_r/A table in the *CISC Handbook* to be 253 MPa.

The axial compression resistance of the gusset is therefore 253 MPa * 15.8mm x 228 mm = 911kN (ample for our 206kN compression). If it was felt that the unbraced length was too short it could be increased in this example without having to increase the plate thickness.

Use a use 15.8mm (5/8-inch) gusset plate.

15.18 PREVENTING BRACE FRACTURE AT THE BRACE CONNECTION

To keep stable brace behaviour it is desirable to prevent the brace from tearing at the face of the gusset plate. S16-14 contained new provisions to address this issue including modifications to the brace connection requirements given in clause 27.5.4 and the calculations for the net section in clause 12.3.3.4. Under some circumstances it will be necessary to add side plates to the brace bay to compensate for the removed material where the HSS is slotted to accommodate the gusset plate. Slotting the HSS to accommodate the gusset plate removes material from the HSS and reduces the section area of the HSS by the thickness of gusset plate as well an allowance for fit and tolerance.

Allowing a 2mm gap for the HSS to slip over the gusset plate the net area of the brace at this point is A_g -2*(Gusset Plate Thickness +2mm)*(HSS Wall Thickness). The connection is evaluated using the provisions of Clause 12.3.3.4, using the $A_g(R_yF_y)$ forces. When evaluating net section fracture of an HSS Clause 27.5.4.2 allows us to multiply the net section resistance by R_y / $Ø_u$ where Ry in this case is not allowed to exceed 1.2 and $Ø_u$ is the resistance factor for ultimate strength and is defined as 0.75 in clause 12.

EXAMPLE—COMPUTE NET SECTION RESISTANCE FOR AN HSS SLOTTED OVER A GUSSET PLATE

Compute the net section resistance at the gusset plate for an HSS 102x102x4.8 brace, the gusset plate is 16mm. Tension-connection force = T_u = 823kN (823kN being $A_g*R_yF_y = 1790mm^2 * 460 MPa = 823.4 kN$

Solution:

The slot in the gusset plate will be 16mm + 2mm = 18mm. The net area of the brace at the 18mm slot for the gusset plate is:

$A_n = 1790mm^2 - 2*4.8mm*(18mm) = 1617mm^2$

By Clause 13.2 $T_r = Ø_u A_{un} F_u$ where A_{un} for an HSS slotted over a gusset plate is now given in 13.3.3.4

$A_{un} = A_n$ when / $L_w > 0.1$ and is less than A_n when / $L_w \le 0.1$. Where L_w is the length of weld on the HSS and is the distance to the centroid of the cut half of the HSS to the face of the gusset. Conservatively taking x' as the distance from the center of the wall of the HSS to the face of the gusset gives an x' of 102/2 – 16/2 – 4.8/2 = 40.6 so L_w=40.6/0.1 = 406mm.

If we use a L_w=400mm weld we get a resistance of $T_r = Ø_u A_n F_u (R_y / Ø_u)$ where in this case R_y cannot exceed 1.2

R_y=460 / 350 = 1.31 use 1.2

$T_r = A_n F_u R_y$ = 1617mm² * 450MPa * 1.2* 0.001 = 873 kN which exceeds our load of 823 kN and no reinforcement is required.

If, however we decided that we wanted to only have a weld length L_w = 225mm then our resistance calculation is more complex.

First step check that L_w is sufficient weld to transfer the 823 kN if a 6mm weld is used and would not want to increase weld beyond 6.4mm on the 4.8mm wall.

Resistance = 4 * 225mm * 0.933 kN /mm = 839 kN (OK)

By 12.3.3.4 we get

we get

$T_r = Ø_u A_{ne} F_u (R_y / Ø_u) = A_{ne} F_u R_y$ = 1486 * 450 * 1.2 * 0.001 = 802 kN which is less than our 823 kN and therefore reinforcement is required.

The problem could be solved by shop-welding flat bar to the sides of the brace in the connection region such that the net area at the gusset plate slot always exceeds the A_g of the brace as shown in the dotted line in Figure 15.19. Another solution is to weld the HSS and the gusset plate together and then weld flat bars or angles to both gusset and HSS and having these extra bars go beyond the gusset plate slot. The latter solution is less popular as it involves more field welding.

EXAMPLE—DETERMINE THE RESISTANCE IF TWO SIDE PLATES ARE ADDED TO OUR PREVIOUS EXAMPLE

Determine the net section resistance if two 50 x 6.4 plates are welded on the sides of an HSS 102 x 102 x 4.8 with T_u force of 823kN with the brace connected to a 16mm gusset plate with a weld length of 225mm.

Solution:

The resistance of the slotted brace is found in the previous example to be 802 kN.

If we add two 50mm x 6.4mm plates our net section resistance will increase to:

Resistance of each plate = (Ø = 0.9) (50mm)(6.4mm) (350,000 kN/m²) (1x10⁻⁶) = 101kN

Or $= Ø_u A_{ne} F_u$ = 0.75 (50mm)(6.4mm)(450,000 kN/m²) (1x10⁻⁶) = 108kN

Use the lesser of these though it might be argued that using the net section fracture would be more appropriate if we are looking at net section failure.

Net section resistance = 802kN + 2 * 6.4 * 50 x 10⁻⁶ * 350,000 * (Ø = 0.9) = 802kN + 2(101kN) = 1004kN

Net section resistance of 1004 kN exceeds load of 802 kN (OK)

Note that the resistance of the side plates are computed with a Ø factor applied and this resistance is not multiplied by $R_y/Ø_u$, as the side plates will not be from the same steel as the brace. The calculation above is conservative and protects the brace against failure.

Shear lag may govern the design of the welds for connection of HSS members. The Code requires the weld length to be a minimum of twice the distance measured along the member between welds. Tests performed by Korol (1996) show that this weld length is conservative.

Where tension-compression HSS braces cross in the centre of cross-braced bays, side plates should connect the braces such that the axial force can be transferred without creating a hinge in one or both of the compression members. Tension-only braces having no brace compression capacity may be connected by gussets at the centre while braces for tension-compression braces should be connected by plates each side as this connection works better for brace systems taking compression.

As structural designer, it is necessary to detail the connection of these brace elements. It is not sufficient to give a connection force of $(R_y F_y)A_g$ and leave the design to the steel detailer. Figure 15.19 shows the detail at the end of a brace where it connects to a single centreline gusset plate. The side plates shown

dotted are added to eliminate the problem of net section failure at the brace end. Under some circumstances they may not be required under S16.

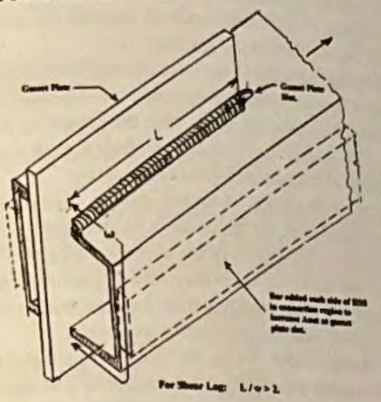


Figure 15.19. Brace showing welded flat bar (shown with dotted line)

Figure 15.20 shows details where the braces cross in a cross-braced system. The side plates must carry the capacity design connection force.



a) b)

Figure 15.20. Tests have shown that using two side plates as shown in (a) (left) is more ductile than using a single gusset plate at the centre connection as shown in (b) (right)

15.19 SEPARATING THE BRACE FROM THE STUDS

To minimize space requirements and architectural impact, the braces are usually buried in the interior or exterior walls of the building. Often these walls will be constructed with 152mm studs limiting the brace member size to 127mm when allowance for tolerance is included. In pre $A_g F_y$ design, a support track for the steel studs was added above and below the inclined brace member and steel studs were inserted above and below the brace (as illustrated in Figure 15.21a). This meant that the brace elements were acting as wind girts and had to be designed for bending moments and have a minimum moment of inertia to reduce deflections (as illustrated in Figure 15.21b).



a) b)

Figure 15.21. a) The brace bay has steel stud track attached above and below the brace. The studs are attached to it. b) The brace bay uses details to separate the studs from the braces

If the brace was located in an exterior wall and the studs were attached to the brace, it would have to be designed for the effects of the wind load perpendicular to the plane of the brace including bending and deflection. The brace bay shown in Figure 15.21b uses details to separate the studs from the braces. The braces are therefore not controlled by out of plane wind loading.

Designing the brace as a wind girt often resulted in out-of-plane wind becoming the controlling factor and producing a brace size in excess of that required for seismic force. To eliminate the requirement to oversize the brace for wind it is necessary to have the studs span the ground or floor below to the beam above without resting on the brace. This can be achieved by replacing each stud with a pair of studs screwed back to back with a cut-out each side to accommodate the brace.

With the introduction of protected zones in S16-09, there was now an additional argument for not connecting the studs to the braces. The protected zone for the brace is defined under S16 as including the full length of the brace and the end connections of the brace. In the protected zone attachments that introduce notches or stress concentrations are not permitted as these may lead to premature failure of the element. While arc-spot welds are permitted these would typically not be used on stud to brace connections and the usual mechanical connectors of screws or shot on pins are not permitted.

Another solution is to cut out one flange of the stud at the brace location and then use a section of track or bridging screwed to the stud to provide continuity for the severed stud flange. Similar steel stud details have been used to accommodate pipes in walls, as the pipe is obviously not designed to carry wind loads. Sample steel stud details are illustrated in Figure 15.22.

Separating the steel studs from the brace requires that appropriate steel stud details be included on structural drawings even if the structural engineer is not responsible for steel stud design.

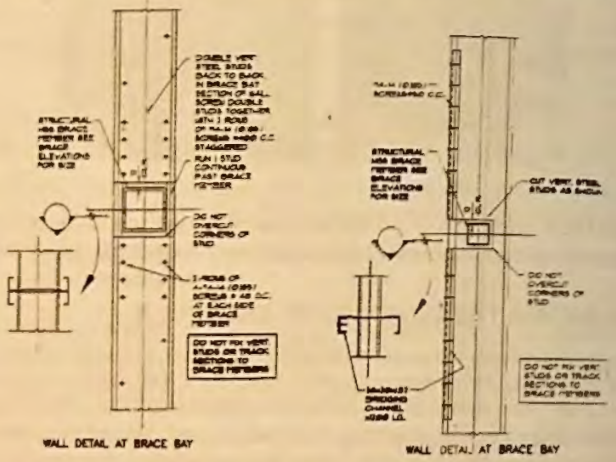


Figure 15.22. Steel stud details to avoid placing wind load on the brace elements

The details shown in Figure 15.22 should also be used on interior walls as well as exterior walls as it prevents fastening to the protected zone and will result in a more rigid wall especially where thin braces are involved.

15.20 CONNECTION TO THE FOUNDATION

The connection of the brace to the foundation produces several problems as the capacity design forces must be transmitted in a rational manner. If we attempt to do this by the traditional bolt formula we ignore the effect of bolt bending in the grout layer as shown in Figure 15.23.

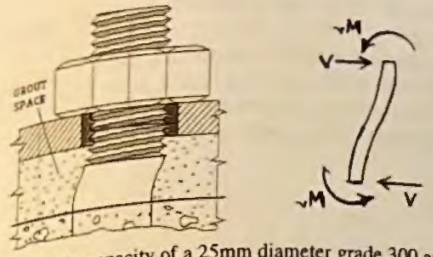


Figure 15.23. Bolt connection to foundation

If we examine the capacity of a 25mm diameter grade 300 anchor rod to resist bending, we find that the bending moment capacity is 0.414kNm/bolt. As required by Clause 26 this capacity is computed using S rather than Z. If we assume the distance between the points of fixity is as follows:

Base Plate	20mm
Grout Layer	25mm
Half embedment of 5 diameters	48mm
	93mm

The maximum shear is $V_r = 2*M_r$ / (Distance between points of fixity)

With a M_r of 0.414kNm for a 25mm bolt this gives us a shear capacity of 8.9kN/bolt – far below the capacity required to resist our capacity design forces.

A shear friction approach will yield greater capacities but usually only sufficient to resist the shear forces on very lightly loaded brace bays.

A more certain method of resisting the brace bay forces is to separate the resistance of shear and tension forces resisting the shear forces by embed plates and angles that can carry the shear force through the grout layer to the foundation much more directly than by bolt bending. Such a system is illustrated in Figure 15.24.

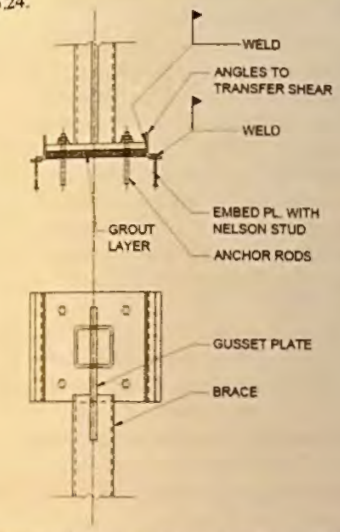


Figure 15.24. Embed plates and angles for foundation connections

In foundation connection details shown in Figure 15.24 the tension force is carried by Dywidag bars acting as anchor rods, which can carry a higher tension than A307 bolts as the Dywidags have a higher yield. There are several grades of Dywidag and the grade selected should be appropriate for seismic anchors and, if the anchor is to be outside, should be appropriate for cold temperatures. The Dywidag bar also has a

raised thread so it is not necessary to reduce its capacity for the net area at the cut threads and the tension capacity is therefore based on gross area using the formula: $T_r = \emptyset_{ab} A_b F_y$.

The shear force is carried by angles that are field welded to both the base plate and embedded bars with attached Nelson studs. Only the outside edge of the angle is welded so that it does not attempt to carry tension forces that would reduce the shear capacity of the Nelson studs. It is necessary to include details for connection between the brace and foundations on structural drawings.

15.21 FOUNDATION CAPACITY

The connection of the braces to the foundation requires that the foundation resist the uplift and shear from the brace. The brace connection forces from the braces undergoing probable tension resistance (T_u) or probable compression resistance C_u must be transferred to the foundation or structure below. For braces attached to concrete foundation elements, uplift under the brace tension can be a problem. A provision of the Building Code is often employed which limits the foundation to resisting $R_dR_o = 2.0$ or $R_dR_o = 1.3$ forces, depending on whether the foundation can be described as an isolated pad or as structure.

The structure can include any structure such as grade beams or crawl space walls, which will not be inclined to dissipate energy by stamping in the same manner as an isolated pad footing. It might be argued that the capacity of the brace itself might be limited by the uplift capacity of the foundation. If, for example, the isolated foundation pad was designed for an uplift force corresponding to $R_dR_o = 2.0$, why should the axial force in the brace not be limited to this uplift? It must be remembered, however, that the seismic forces are dynamic. As a result, our single-storey brace will have a period in the 0.1 second range. The axial force in the brace will go from zero to full tension in one-quarter of this cycle or 25 one-thousandths of a second. Trying to lift the foundation in that instant will result in axial forces in the brace considerably higher than those required to lift the foundation very slowly.

The foundation "bailout" forces provided by the concrete code would be beneficial for use in design. See also Building Code Clause 4.1.8.16.1, which allows footings that are able to rock to be designed for $R_dR_o = 2.0$. The provisions for footings that are not capacity protected are covered in depth in A23.3-14 with extra provisions to increase the computed deflections for the structure.

Note that the resistance of all other elements in the brace bay is to be designed to be sufficient to resist the capacity design forces (or possibly limited to the forces computed using $R_dR_o = 1.3$). We do not limit the connection forces on elements connecting to the foundation weight as these elements are subject to rapid loading and impact that will put loads on the brace exceeding the static dead load weight of the footing.

We use the $R_dR_o = 2.0$ for the foundation bailout on $R_d = 1.5$ to $R_d = 3.0$ brace bays. However, there does seem to be a misconception that you can extend this bailout backward and design the connection between the brace bay and the footing and even some other parts of the brace bay for the "weight" of the footing. After all, how could the uplift force be greater than the weight of the footing? There are a variety of reasons that the force on the anchor rods is greater than the weight of the footing, including:

- The weight of the footing assumes that the slab on grade will not try to arch and hold down the footing.
- The static weight of the footing alone does not take account of impact loading — a single-storey brace bay is cycling at 0.1 sec so we get only 0.025sec from neutral to full tension. The "dynamic" weight of the footing will be considerably higher than the static weight resulting in impact factors that will be ignored by using just the static weight of the footing.
- The weight of the footing alone does not take account of possible weight added over footing due to storage or having the footing depressed with greater overburden etc.
- The weight of the footing alone does not take account of the side friction or bottom suction on the footing. This is soil dependent but to ignore it is not appropriate.

It would not be inappropriate to add a note to the S16 commentary (and possibly NBCC commentary) to state that while the brace bay foundation could be reduced in some circumstances to $R_dR_o = 2.0$ it is not appropriate to reduce the capacity of anchorage and other parts of the brace bay to match the static weight of the footing.

The A23.3 concrete code footing "bailouts" were increased in a 2009 revision to $R_dR_o = 2.6$ if the footing is allowed to rock but these provisions do not comply with the Building Code and should not be used.

We refer to these as "bailouts" as these limiting forces are where the code gives us an opportunity to cease our capacity design approach and use limiting or "bailout" forces.

There are several places in the steel and concrete standards as well as in the Building Code where limiting forces are prescribed. For S16 Clause 27.1.2 tells us that the design forces need not exceed those corresponding to $R_dR_o = 1.3$ acting on the structure. This provision is only required to be exceeded in some areas of connection detailing (especially in moment frames) and in cantilever structures.

The bailouts are an interesting problem. Basically they come down when you say "Enough is enough." The uplifts from capacity design can be huge and are may be well in excess of the "elastic" response. At some point the Code says it is acceptable to cut off the forces as reasonable safety has been achieved. NBCC (see Clause 4.1.8.15(7) bailout at $R_dR_o = 1.3$ provided the referenced design standards (e.g., S16) permit this bailout.

From the standpoint of footing uplift in short period buildings, we are trying to pick up the footing in a fraction of a second, something that does not get appreciated in a "static" analysis. In the example, our footing uplift of 268kN requires a footing of about 11.4 cubic metres of concrete the footing will be about 3.25m x 3.25m x 900mm deep with 300mm of soil overburden. A footing of this size is not going to be picked up in 0.025 seconds (period of building divided by 4) without exerting a lot more resistance than its 268kN of dead weight.

EXAMPLE—DETERMINE FOUNDATION FORCES

Determine the foundation forces on the sample brace bay footings the footings are able to rock.

Solution: The brace bay is shown in Figure 15.25 along with the forces under $R_dR_o = 2.0$ conditions the compression brace can carry C_u or C'_u. The $R_dR_o = 2.0$ forces are less than the capacity design forces will be used for foundation design. If it is decided to tie down the footing using soil anchors or to connect the footings together using a grade beam then $R_dR_o = 1$ forces should be used. Note that the force on the brace bay at $R_dR_o = 2.0$ cannot be computed by just multiplying by the ratio of R_dR_o value that the brace force was calculated on divided by the rocking R_dR_o of 2.0

Forces on brace bay at $R_dR_o = 2.0$: (Dead and live or snow loads need to be added to this.)

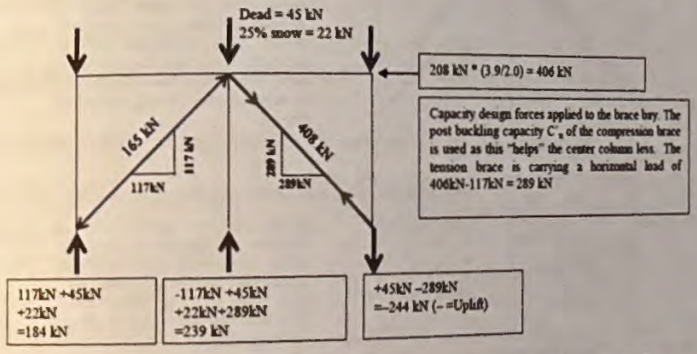


Figure 15.25. Brace bay with $R_dR_o = 2.0$ foundation bailout forces applied

The uplift force of 244kN is considerable and represents 244kN / 23.5kN/m^3 = 11m^3 of concrete. At 1m deep this is 3.3m x 3.3m and some designers will choose to replace this with tie-down anchors. If during the design of the foundation you decided to go to tie-downs, then it would then be necessary to use $R_dR_o = 1.0$ forces on the brace bay for designing the footing uplift as the system cannot stamp around and dissipate energy.

15.22 PROTECTED ZONES

Protected zones were introduced in S16-09 as a way of preventing the introduction of harmful stress risers on the yielding elements that will lead to their premature failure. Clause 27.1.9 tells us that we are not allowed to have connections to the protected zone, including mechanical fasteners for deck or stud tracks and the drilling or cutting of holes in the protected zone is also not permitted. The protected zone must be shown on both the structural drawings and reflected in the shop drawings. The protected zone is given in the clauses for each system. For moderately ductile concentrically braced frames the protected zone is given in Clause 27.5.6 and requires that both the brace and any elements that connect the brace to beams or columns be designated as protected zones.

15.23 REDUCING THE NUMBER OF BRACE BAYS

One factor that becomes apparent, especially if the braces are lightly loaded, is that a more efficient design can be achieved by reducing the number of brace bays, thus increasing the force on each brace bay. While this will produce a saving in the brace bays, it requires that more provisions be made to have the seismic forces travel farther before they reach brace bays. In our sample problem there is only one brace on each side of the building so reduction of the number of brace bays is not an option.

15.24 TENSION-ONLY BRACING SYSTEMS

One solution to reducing the connection forces that have to be accommodated is to use a tension-only bracing system and reduce the size of the brace members to the point where their tension capacity is the controlling factor. The difference between a tension-only system and a tension-compression system is that when the load on the brace bay under the $R_dR_o = 3.9$ forces and the notional loads are distributed to the braces for sizing the brace, all the force goes to tension elements while in tension-compression systems it is divided equally between tension and compression elements and the braces must be sized to carry the compression loads as shown in Figure 15.26.

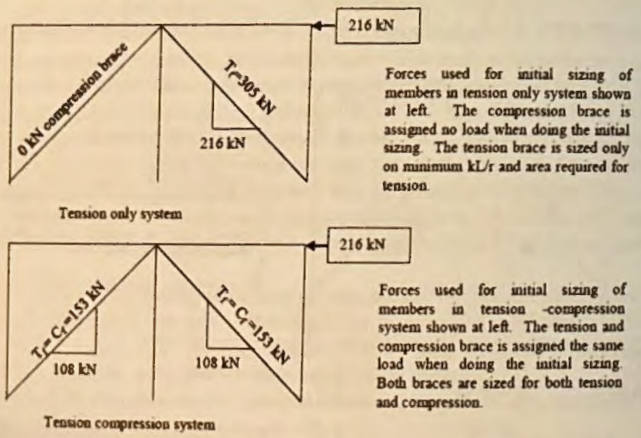


Figure 15.26. Comparison of tension-only and tension-compression systems

Slenderness of the brace members is still limited by satisfying of 27.5.3.2 that limits the value of kL/r to be less than 200 for $R_d = 3$ braces but when using one- and two-storey tension-only $R_d = 2$ braces, Clause 27.6.3.1 allows us to reduce the kL/r value to 300. The minimum kL/r requirement in tension-only systems

is to increase the ductility of the system as well as to reduce the undesirable service conditions such as sag and vibration.

EXAMPLE—DESIGN A TENSION-ONLY BRACE

Take our single-storey brace designed earlier as tension compression and design as an $R_d = 3.0$ $R_o = 1.3$ tension-only brace. The brace bay height is 3.5m and the overall width is 7.0m with two inclined concentric braces is carrying a factored seismic shear of 208kN and notional load of 8kN.

Solution: With a brace length of 4.95m the minimum radius of gyration that is acceptable is 4950 / 200 = 24.7mm. The tension load in the brace is 216kN / 0.707 = 305kN as shown in Figure 15.27.

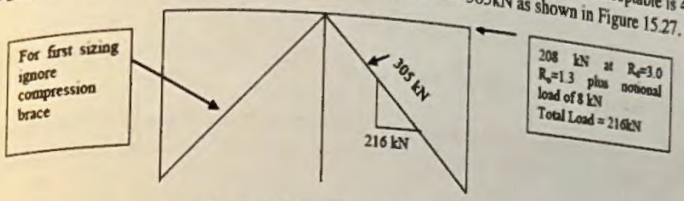


Figure 15.27. Tension load in the brace

Using the column capacity tables (green pages) of the *CISC Handbook*, we see that an HSS 76x76x4.8 has a tension capacity 413kN and a radius of gyration of 28.8mm.

Check KL/r: KL/r = 1.0x4950/28.8 = 172 (OK <200)

Check b/t: b/t = (76 – 4 * 4.8) / 4.8 = 11.9

(It might have been suggested that we use an HSS 64x64x4.8 as this has a T_r of 334kN unfortunately the kL/r ratio for this section is 210 which exceeds the maximum of 200)

Maximum permitted (Clause 27.5.3.2):

At KL/r = 100 get 17.6

At KL/r = 200 get 22.4

Linear interpolate for KL/r = 172 get max b/t = 21.1 OK (b/t of 11.9 is less than this)

Computing brace capacities and factored resistances, we get the data shown in Table 15.6.

Table 15.6. Brace capacities and factored resistances

Condition	Symbol	Calculate	For our brace
Factored Tension Resistance	T_r	$\phi A_g F_y$	413 kN
Factored Compression Resistance	C_r	Use green pages	73 kN
Probable tension resistance	T_u	$A_g R_y F_y$	603 kN
Probable compression resistance	C_u	Lesser of: $1.2*C_r/\phi$ computed with R_yF_y or $A_gR_yF_y$	100 kN
Probable Post Buckling Compression Resistance	C'_u	Lesser of: C_r/ϕ computed with R_yF_y or $0.2A_gR_yF_y$	81 kN

We can now put these forces on the brace bay and determine the overstrength, this is illustrated in Figure 15.28.

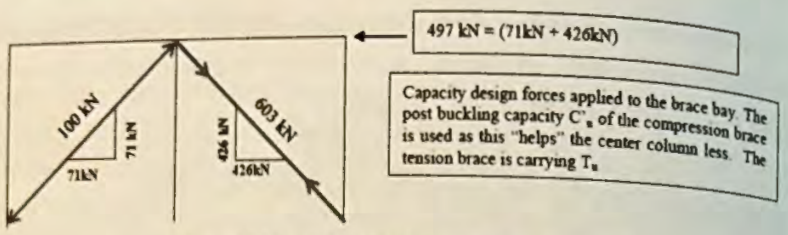


Figure 15.28. Free body diagram of brace bay with brace members carrying their capacity design forces

Effective overstrength of brace = 497kN / 208kN = 2.39

With this bracing scheme the capacity design force for diaphragm design would be 497kN which is considerably less than the 748kN with the tension-compression system. The resulting member size is shown in Figure 15.29.

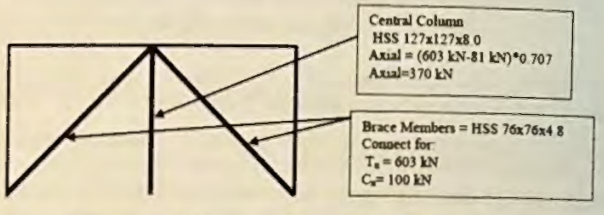


Figure 15.29. Selected brace members R_dR_o = 3.9 tension-only system

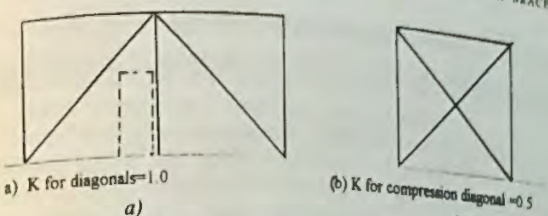
The probable tension capacity of 603kN in the tension-only brace is considerably more efficient than the probable capacity of 823kN found for the tension-compression system. For braces with the low forces found on many single-storey buildings, using a tension-only system will reduce the design shears on the roof diaphragm. Many engineers believe that a tension-only bracing system is less ductile than a tension-compression system, in part due to the smaller sizes and resulting earlier onset of brace buckling in the tension-only system.

15.25 THE CROSS BRACED SYSTEM

The cross braced bay has the advantage that under lateral loading the tension diagonal can provide lateral restraint to the compression diagonal and increase the load at which buckling of the compression diagonal occurs. Using the complete length of the diagonal, the value of K of the compression brace supported at the midpoint by a tension diagonal is shown by analysis to be 0.5. For further discussion of the effective length of compression braces of cross-braced cranes, see the two AISC references given.

15.26 CONFIGURATIONS OF TENSION-ONLY BRACE BAYS

Figure 15.30 shows two alternatives for tension-only brace configurations. The alternative selected will depend partly on architectural constraints, while it is possible to place a doorway in the option at the left (Figure 15.30a); the option at the right (Figure 15.30b) uses only half the wall length. The cross-braced option shown in Figure 15.30b will interfere with doors, windows and in wall services such as plumbing lines in only one bay.



a) *b)*

Figure 15.30. Two alternatives for tension-only braces

15.27 TWO-PHASE BRACING

If the bracing system is designed as tension only, then there will often be economies in terms of the forces to be designed for. However it may be possible to design a tension-only system that works as a tension-compression brace for low-level earthquakes and wind. If brace size can be selected so that the KL/r ratio is below 200, then the compression capacity of the brace system can be counted on to resist minor lateral loads, even if the compression capacity of the brace is not sufficient to resist the full force of the design earthquake. Braces that are designed to resist minor but more frequent lateral forces as tension-compression braces, while resisting larger forces such as the design earthquake as tension-only braces, can be referred to as two-phase braces. It should be noted that there is no requirement in the Code for tension-only concentrically braced frames to carry any percentage of their load by compression diagonals. Brace bays using rods that act almost exclusively in tension have been used successfully in many light buildings, but it is beneficial to have some compression capacity in the brace system such that wind and other routine occurrences will cause buckling and resulting lateral movement of the brace members.

An example where a two-phase cross brace is analyzed is shown in Figure 15.31. The brace-bay is analyzed as tension only but the configuration of the brace allows it to take a reasonable horizontal load before the onset of buckling of the compression brace.

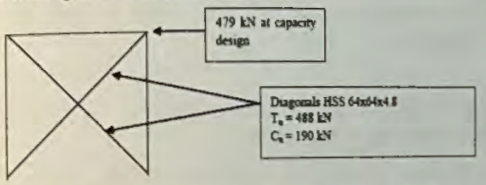


Figure 15.31. Resulting member sizes and connection forces in two-phase cross brace

Resulting R_d = 3.0 R_o = 1.3 Systems designed for 208kN, are shown in Table 15.7.

Table 15.8. Resulting brace configuration

Brace configuration	Brace Size	Tension Connection Force (T_u)	Brace-bay Horizontal load capacity at over-strength	Brace-bay Horizontal load capacity when first brace reaches C_r
Tension-Compression	HSS 102x102 X 4.8	823 kN	748 kN	212 kN
Tension Only	HSS 76x76x4.8	603 kN	497 kN	103 kN
Cross Braced (Two phase)	HSS 64x64x4.8	488 kN	479 kN	188 kN
R_dR_o = 1.3 Maximum			624 kN	

15.28 DESIGN OF A TWO-LEVEL BRACE SYSTEM

Returning to our original problem from Chapter 14, we have the forces on the brace as shown in Figure 15.32.

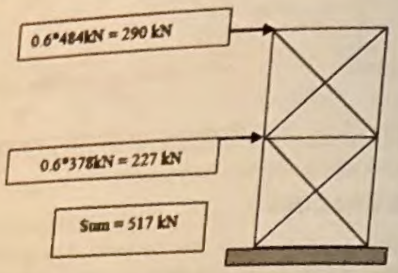


Figure 15.32. Forces on a two-level brace

The brace bay can be designed as two separate lifts. The columns must be designed for the capacity design forces from all the braces yielding at the same time. A method that was once proposed for reducing the load from three-storey or taller brace bays has now lost favour. The braces for the lower floor should be as strong as or stronger than the braces of the floor above. However, it is not necessary to have the lower-storey brace bay have the factored resistance equal to the seismic forces plus the capacity design forces from the upper floor.

The design sequence is similar to that for a single-storey brace:

a) Find the design forces on the brace bay at each level, including the effect of eccentricity and notional loads.
b) Size the brace elements for the forces in (a), keeping in mind restrictions on kL/4 and b/t.
c) Determine the probable connection forces C_u, C'_u, and T_u.
d) Put the probable connection forces in the frame as we did for the single-storey brace and design columns and foundations.

15.29 VERIFYING THAT THE SEISMIC SYSTEM SATISFIES CODE REQUIREMENTS

A fundamental step in designing a seismic system is to make certain that the detailing satisfies all the Code requirements for the ductility value R chosen.

It is necessary for the system to fully satisfy ALL clauses that apply and are appropriate for the system chosen. If certain clauses cannot be satisfied, then a lower value of the force modification factor R_d must be used where and all of the requirements must be satisfied for the lower ductility system. When going to a lower R_d-value, the forces for the system are increased to compensate. The aim is that less ductile systems are designed for higher forces and will therefore have less excursions into the inelastic range. For example, a designer chooses to use a moderately ductile concentrically braced frame having an R_d-value of 3.0 and computes seismic forces based on this value. During the detailing phase, it is determined that, to satisfy the constraints of the project at hand, some of the detailing requirements of Clause 27.5 cannot be met. The designer decides to go to a Type LD (Limited Ductile) Concentrically Braced Frame having an R_d-value of 2 as he or she can fully satisfy all the requirements of Clause 27.6. The forces on all brace bays are then multiplied by the ratio of the R_d-values (3.0/2.0 = 1.5).

Note that if it is desired to reduce the R_d value to 1.5 and describe the system as "Conventional Construction," then there are restrictions in the Building Code that limit the type of building that can be designed with a seismic system using an R_d value of 1.5.

- Building Code Table 4.1.8.9 shows that buildings having $I_E F_a S_a(0.2)$ of 0.35 or greater (most of British Columbia) using "Conventional Construction" are to be a maximum of 15m tall.
- Clause 4.1.8.10 requires that post-disaster buildings have a SRFS with an R_d of 2.0 or greater.
- Building Code Table 4.1.8.9 shows that buildings having $I_E F_a S_a(0.2)$ of 0.35 or greater (most of British Columbia) using "other steel SRFS not defined above" (i.e., they don't even meet the requirements of "Conventional Construction") are not permitted.

It is necessary to detail the adjacent elements to the seismic resisting element to transfer all loads into the seismic element including shear lugs, drag struts and so on. The supports for the seismic element must be designed to resist the capacity design loads from the seismic element. This includes appropriate detailing of the foundation elements and connections to the foundation.

Use caution when using "bailout" values of a force corresponding to $R_d R_o = 1.3$. These bailouts can lead to undesirable failure modes in the case of seismic events greater than the code specified earthquake. Contrary to the belief of many engineers, the code-specified earthquake is not the maximum earthquake that can occur at the site. Instead, the code-specified earthquake has a 2% probability of being exceeded during the next 50 years. There is also a considerable variation in the seismic loading.

With the possible exception of $R_d = 1.5$ systems the detailing requirements for any seismic system is complex. The designer is now working with capacity design to make sure the elements that are intended to yield will yield without failure of adjacent elements in the load path. The designer must also work to make sure that the yielding element can indeed yield over several cycles without degrading. Several years ago the requirements in the steel standard were a lot simpler to follow but now it is necessary to develop checklists for the system being used by going through the Code on a clause-by-clause basis. An example of such a checklist is shown in Table 15.8.

Table 15.8. Checklist for $R_d = 3$ $R_o = 1.3$ concentrically braced bays under S16

TENSION ONLY SYSTEM with HSS Brace Members Higher Seismic Zone		
PROJECT:		
DATE:		
ENGINEER:		
Clause	**Requirement**	**OK?**
27.1.1	Minimum seismic forces calculated in accordance with NBCC 4.1.8	
27.1.2	Capacity design including diaphragm and collector elements.	
27.1.3	All elements in load path shown on structural drawings	
27.1.4	Non-seismic elements able to carry load in deflected position	
27.1.5.1	Steel G40.20 / G40.21 or ASTM A992 Fy<=350MPa NO ASTM A500	
27.1.5.2		
27.1.5.3	Welds of primary elements of seismic system to have filler material with Charpy impact test value of 27J at -30deg C (Call up in general notes and brace bay notes)	
27.1.6	Bolted assembly have pre-tensioned bolts. (Call up in notes on drawings)	
27.1.6	Bolted assembly use clean mill scale or better on contact surface (Call up on drawings)	
27.1.6	No long slot holes for bolted connections	
27.1.6	Short slot bolt \holes only if perpendicular to load.	
27.1.6	Two bolt diameter clear at end of bolt if bearing load >0.75Br	
27.1.7	Use $R_y F_y$ for yield stress $R_y F_y$ =385MPa for G40.21 350W and 300W	
27.1.8		
27.1.9	Define "Protected zones" and show on drawings	

TENSION ONLY SYSTEM with HSS Brace Members Higher Seismic Zone		
PROJECT:		
DATE:		
ENGINEER:		
27.5.2.1	No knee bracing or K bracing – pairs of braces or single braces do not meet one side of column between floors	
27.5.2.2	Tension braces in both directions with capacity 0.75 to 1.33 of compression braces	
27.5.2.5	<=20m in height – increase forces if over 16m high	
27.5.2.5	Fully continuous columns	
27.5.2.5	Column splice restrictions for buildings with greater than 2 storeys. (Satisfied for 1- or 2-storey column with no splices.)	
27.5.3.1	KL/r for brace min of 200	
27.5.3.2	For KL/r <=100: Rect. and sq. HSS b/t<17.6 Round HSS b/t<28.6	
27.5.3.2	For KL/r <=200: HSS to be Class 1	
27.5.4.1	Minimize eccentricity in connecting elements	
27.5.4.2	Connect for $A_g R_y F_y$ in tension bailout at R_d R_o =1.3 on system	
27.5.4.3	Provide rotation capacity in connection.	
27.5.5.1	Design columns for brace force and gravity load and $0.2ZF_y$	
27.5.5.2	Columns continuous over min 2 storeys in multi-storey and constant cross-section.	
27.5.5.3	Partial joint penetration welds in columns have restrictions	

15.30 RELAXATIONS FOR R_d = 2.0 LD (LIMITED DUCTILITY) CONCENTRICALLY BRACED FRAMES (CLAUSE 27.6)

Before the arrival of S16-01, many designers of low-rise steel buildings used R = 2 nominally ductile systems as the detailing requirements were less, and for many low-rise buildings, the use of a more ductile system was of no benefit as capacity design led to designing for similar loads as the less ductile system. With S16-01(R05) detailing the less ductile R_d = 2 system requires meeting all the criteria of the moderately ductile system with relaxations provided in Clause 27.6. This is illustrated in the flowchart in Figure 15.33.

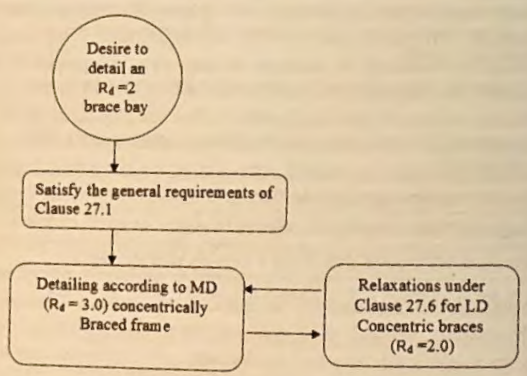


Figure 15.33. Flowchart for brace bay design

The relaxations may not be very useful to the designers of low-rise buildings in higher seismic regions. For areas where $S_a(0.2)>0.35$, the following are the only relaxations that can be used when going to an R_d = 2 system.

A) For concentrically braced tension-compression with HSS braces:
   a) The maximum height of an R_d = 2 concentrically braced frame can be 60m in height instead of 40m in the R_d = 3 system (27.6.2.1)
   b) There is no item b.

B) For concentrically braced frames with tension only bracing systems and HSS braces:
   a) The maximum height of an Rd = 2 concentrically braced tension-only system can be 60m instead of 40m in the Rd = 3 system (27.6.2.3)
   b) In one- and two-storey structures only the tension brace can go to a kL/r of 300 instead of a maximum of 200 in the Rd = 3 system.
   c) When the brace slenderness exceeds 200 we no longer have limits on b/t for the braces.

Many designers will find that relaxations are of little assistance and will choose to design only R_d = 3.0 systems. Other designers will choose to design only R_d = 1.5 systems where the detailing rules are left more to the designer however there are issues with this approach that we will discuss in Chapter 19.

15.31 INTRODUCING A FUSE IN THE BRACE

After $R_y A_g F_y$ connection design and capacity design principles became common in brace bay design, several designers conceived of placing fuses in the brace such that the force in the brace was limited. Several different schemes were examined from cutting holes in the brace to cutting the brace off and replacing with an area such as plates or angles that have lower axial resistance. Several practising structural designers and several researchers suggested possible fuses and, in 1999, a series of cyclic tests on these schemes were performed at Montreal's École Polytechnique and Vancouver's University of British Columbia. The results of these tests were not encouraging. The fuses all failed prematurely with a local failure, while some were able to survive a few cycles. None of them illustrated the stable cyclic load-carrying capacity behaviour that we are looking for in our seismic force resisting systems. Designers who are very anxious to use brace fuses in their designs would be better off using buckling restrained braces. If you are determined to use a brace fuse, then it should be full scale tested to demonstrate stable cyclic behaviour.

15.32 CHEVRON BRACE FRAMES

Chevron brace bays are concentrically braced frames that have a configuration that allows for doors through part of the bay width but there are additional requirements that must be met particularly for the beam. Braces can be attached to the beam either from above or below but the most typical is to have the braces intersect the beam from below. This configuration is shown in figure 15.34.

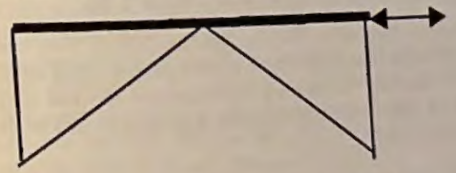


Figure 15.34. Chevron-braced system with braces intersecting the beam from below

The chevron brace beam has various requirements that must be satisfied for the most ductile chevron brace (R_d=3 chevron braces covered by Clause 27.5), as follows:

- Beam must be continuous between the columns.
- The braces must connect without eccentricity in the middle half of the beam.
- There is an advantage to having a class 1 beam, especially in structures that are less than four stories in height where S16 permits a reduction in the tensile brace force to $0.6T_u$.
- Beam must have top and bottom flange bracing at the point where the chevron braces connect. The brace shall be designed for 0.02 times the beam flange yield force.
- The forces to be considered as acting on the beam where the braces are connected from below are shown in figure 15.35 and under these forces there is no vertical support considered from the braces.
- Effect of axial load that occurs in the beam under seismic loading should also be considered in conjunction with the moments from both gravity and seismic actions.
- Beam-to-column connection forces can be determined using the free body diagram below but if a tension force less than Tu is used the beam needs to be class 1 with the forces increased to cause a moment equal to R_yM_p at the brace connection point in the beam. Don't forget to include any axial load in the beam in the column connection force and show "pass through forces" where necessary.

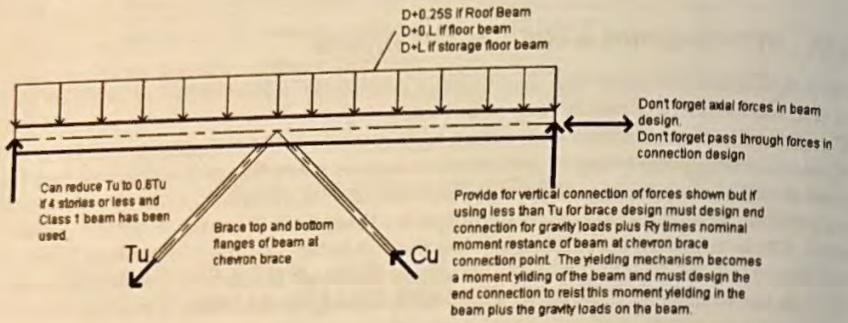


Figure 15.35. Free body diagram of a chevron brace beam

There are additional requirements if the braces connect to the beam from above and there are some reductions in requirements if the brace is an R_d=2.0 chevron brace particularly if the short period spectral acceleration ($I_EF_aS_a(0.2)$) is less than 0.35.

15.33 DUCTILE BUCKLING RESTRAINED BRACED FRAMES – CLAUSE 27.8 (R_d = 4.0 R_O = 1.2)

Provisions for buckling restrained braces were introduced into S16-09. They have been successfully used on projects in Canada. Buckling restrained braces involve placing a steel tension-compression element in a sleeve that prevents buckling. The sleeve is separated from the axial carrying element such that it does not participate in the axial forces but gives system good capacity and stable hysteresis loops in both tension and compression.

CHAPTER 16
SEISMIC DESIGN OF MOMENT FRAMES

The previous two chapters have emphasized the philosophy of seismic design and how knowledge of seismic design is fundamental for an engineer working on steel buildings in the seismic regions of Canada. Earlier chapters have covered seismic loading, diaphragms, and brace bay design using capacity design procedures. This chapter will cover moment frames, which are also designed under the provisions of capacity design, but before we get there we will study "conventional construction." While not strictly following capacity design, conventional construction may be an appropriate way to design some industrial structures.

16.1 MOMENT FRAMES

Moment frames are often used where it is desired to have an open space uninterrupted by brace bays or walls. Prior the January 17, 1994, Northridge Earthquake moment frames were the favoured seismic system in the United States, including the most seismically active regions of California and Alaska. Unfortunately the Northridge Earthquake exposed that welding of the beam-column connection could often be the weak link in the seismic system. In the years since 1994, there has been an extensive testing and analysis program carried out through the Federal Emergency Management Agency (FEMA) and through the universities to try to establish methods of constructing moment frames such that failures will not occur again. The Northridge Earthquake occurred after S16.1-94 was finalized and the problems in moment-frame connections and solutions to those problems were not incorporated in that edition of the Code. Subsequent to the Northridge Earthquake editions of both Canadian and American codes embraced stringent seismic requirements for moment frames and the detailing of moment frames became challenging and these requirements continue to be reflected in S16. Ductile and limited ductile moment resisting frames have two joints that have demonstrated ductility in physical testing. This is a lengthy and expensive process that would be beyond consideration for most projects and the available test data from FEMA has generally covered columns of 350mm or less which are difficult to use with Canadian drift limitations. The use of column sections of 350mm or less will result in very thick column flanges when satisfying strong-column/weak-beam criteria and drift limitations. Thick column flanges require very careful detailing and extensive field inspection to protect against lamellar tearing at the beam-column connection. Moment resisting frames with limited ductility (R_d = 2) are ideal for elements in large glass storefronts and in other low-rise buildings, and nominally ductile moment frames are now permitted in higher seismic risk zones provided they are less than 30m high. The flowchart of the S16 moment-frame options along with the rotational requirements of the joint are shown in Figure 16.1.

The detailing of moment-frame connections for seismic loads is a complex task that involves solving a series of restrictions. CISC has produced a manual, *Moment Connections for Seismic Applications*, which is an essential reference for the restrictions of each moment-frame connection system. These restrictions are a result of ongoing research and testing. During the production of S16-14, the moment-frame guide was rewritten using research that included the AISC work on yield line anaylsis of moment-frame

connections. When doing moment-frame design, refer to the most current edition of the CISC moment-frame guide, which at the time of writing is the 2nd (2015).

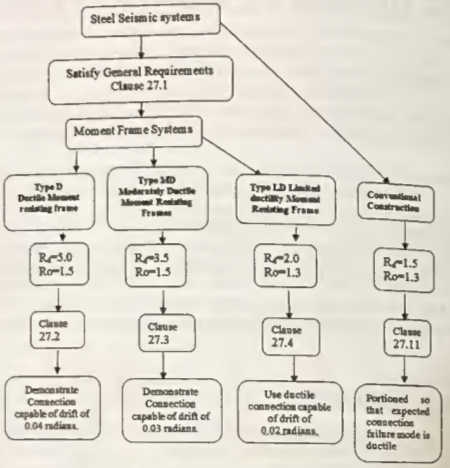


Figure 16.1. Flowchart of the S16 moment-frame options along with the rotational requirements of the joint

16.2 CHANGES TO MOMENT-FRAME DESIGN FOLLOWING THE NORTHRIDGE EARTHQUAKE

In 1994, the design community in California was surprised to find extensive damage to steel moment frames had occurred during the Northridge earthquake. Of particular concern were numerous failures at and near the beam-to-column welds. An extensive laboratory-testing program resulted in improvements in the configurations of beam-column joints and American seismic codes reacted quickly to introduce stricter requirements that would address the observed problems. The methods of addressing the failure of the beam-column welds were to make certain that the weak link in the frame was not the beam flange to column flange weld. These methods fell into two camps. The first was to increase the strength of the beam at the location of the column, a process that involved the addition of bolted or welded plates and stiffeners. Several of these strengthening methods are copyrighted and royalties must be paid to use the system. The second method of eliminating the beam-column weld as the weak element was to deliberately weaken the beam by cutting smooth semi-circular shapes out of the flanges of the beam near the column, a process now known by the somewhat artless descriptive of "dogboning" the beam. As the moment diagram due to gravity loading decreases sharply near the column, it is possible to remove sections of beam flanges near the column face without serious effect on the dead and live load behaviour of the frame. The deliberate weakening of the beam near the column gives a predefined area where plastic behaviour will occur under seismic loading and acts as a protecting fuse to the beam-column weld. Figure 16.2 shows the basic configuration of a reduced beam section (RBS) joint while basic requirements for designing this type of beam connection are given later in this chapter.

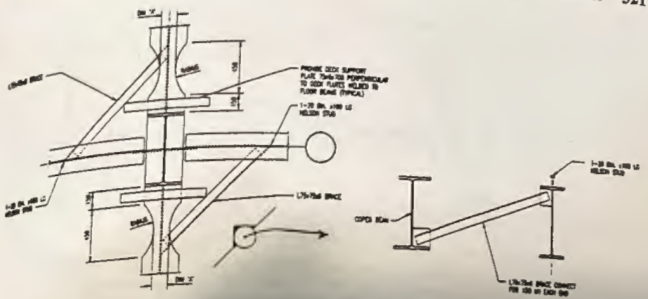


Figure 16.2. Configuration of a beam-column joint using the reduced beam section or dogbone. Most research now indicates no need for the braces to be added if the beams are connected to a floor

Although it may be considered that the use of reduced beam section details in steel moment frames is not common practice in Canada, this situation it likely to change. So far, at least three significant projects in Vancouver have been designed and constructed using a steel moment frame with dogbone protected connections. A reduced section beam-column joint from that project is illustrated in Figure 16.3. Canadian steel fabricators, detailers, and engineers who work on projects in the United States are also finding that the dogbone detail is becoming more common.

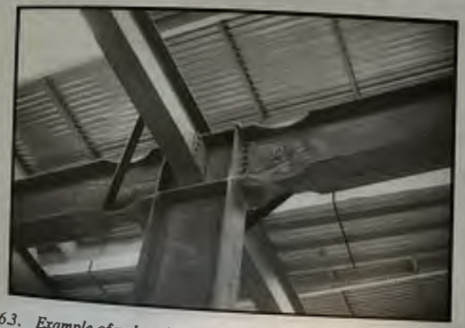


Figure 16.3. Example of reduced beam section joints used on a project in Vancouver

16.3 CERTIFIED OR PRE-QUALIFIED JOINTS AND DRIFT REQUIREMENTS

After the Northridge earthquake there was a realization that moment-frame joints would have to be "certified" with exact recipes that would assure ductile behaviour was achieved. The mantra of the certified joint was the inter-storey drift angle is shown in Figure 16.4, which is taken from FEMA 350.

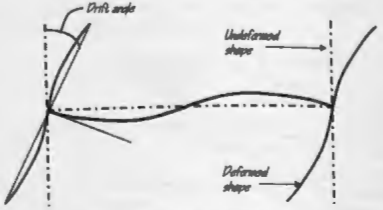


Figure 2-2 Interstory Drift Angle

Figure 2-2 illustrates the interstory drift angle, for a frame with fully restrained (FR) connections and rigid panel zones. Prior to lateral deformation, the beam and column are joined at right angles to each other. Under elastic deformation, the column and beam will remain joined at right angles and the beam will rotate in double curvature between the two columns. The interstory drift angle is measured as the angle between the undeformed vertical axis of the column and the deformed axis of the column at the center of the beam-column joint. For the idealized FR frame with rigid panel zones, shown in the figure, this same angle will exist between the undeformed horizontal axis of the beam and the deformed axis of the beam, at the beam-column connection. In FEMA-273, this angle is termed the chord angle and is used as the parameter for determining beam-column connection performance. However, for frames with panel zones that are not rigid, frames with partially restrained connections, or frames that exhibit plasticity at the connection, the chord angle of the beam will not be identical to the interstory drift angle. For such frames, the interstory drift angle, reduced for the effects of axial column elongation, is a better measure of the total imposed rotation on all elements of the connection, including panel zones and connection elements, and is used as the basis of these Recommended Criteria.

Figure 16.4. Interstorey drift angle
Source: FEMA 350

To satisfy the drift limitation requirements, it is necessary to provide a beam-column joint that exactly matches a tested beam-column joint that has been shown to give the desired inter-storey drift angle. S16 includes Annex J "Ductile moment resisting Connections," which refers the user to several sources of tested joints, including the CISC publication *Moment Connections for Seismic Applications*.

Several tests have been performed to establish tested assemblies but up until recently most have involved columns of depth less than 350mm (W360). This leads to the following issues:

- Having a strong column weak beam is more difficult to achieve as typical moment-frame beams in buildings (which also have to resist dead and live loads) will be deeper than 350mm.
- Trying to increase the moment capacity of the column to exceed the capacity of the beams will often result in very thick column flanges and these are more prone to welding problems.
- The 350mm deep column is difficult to make the Canadian drift limitations work.

Fortunately there is now some test data that shows very favourable results for moment frames involving W610 beams (See Ricles, Zhang, Lu, and Fisher in the References section at the back of this book.)

16.4 STRONG COLUMN, WEAK BEAMS

Central to the philosophy of considering the effect of yielding members on the column forces is to prevent strong beam-weak column behaviour and soft storeys. Figure 16.5 is reproduced from the concrete code CAN/CSA A23.3, and illustrates the problem with soft storeys.

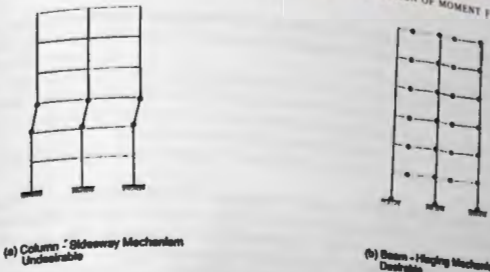


Figure 16.5. Hinge locations for desirable seismic behaviour
Source: CAN/CSA A23.3

Clause 27.2.3.2 of the steel standard also requires that the columns be stronger than beams. The effect of this clause will often be to increase the size of columns.

To examine some of the detailing issues of steel moment frames we will take the two-storey frame that we developed the forces for in Chapter 14 and we will convert it from an $R_d = 3.0$ $R_o = 1.3$ concentrically braced frame to a Limited ductility moment resisting frame with $R_d = 2.0$ $R_o = 1.3$. The transformation is shown in Figure 16.6.

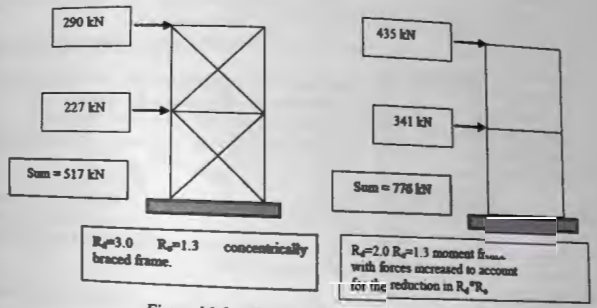


Figure 16.6. Forces for sample moment frame

The basic requirements for Limited ductility ($R_d = 2.0$) moment frames in a higher seismic risk area such as Vancouver or any other location where the short period spectral acceleration ratio is greater than 0.55 is as follows:

- Limit to 30m in height when $I_E F_a S_a(0.2) \geq 0.75$ and to limit to 60m when $I_E F_a S_a(0.2)$ is between 0.35 and 0.75. There is no limit on height when $I_E F_a S_a(0.2)$ is less than 0.35
- Beams to be Class 1 or 2.
- Columns to be Class 1 and I-shaped. (This eliminates HSS columns in moment frames.)
- When $I_E F_a S_a(0.2)$ is greater than 0.55 we must satisfy the "Strong column weak beam" requirements of 27.2.3.2 but can use the beam plastic moment when computing the moments applied to the column. $M_{pb} = F_y * Z$ (No Ø, No R_y for $R_d = 2.0$ frames).

- Must not have a "weak storey." The storey shear resistance must be at least as great as that above. This might be a challenge if the ground floor has a greater floor-to-floor depth than the floor above.
- Check that the joint panel shear is not exceeded in the same way as ductile moment frames and add web stiffeners if the column web shear capacity is exceeded and add web stiffeners if there is insufficient strength. As web stiffeners require a lot of cutting and welding it may be more efficient to use a column section that has a thicker web to start with.
- Beam-column connections are given in Clause 27.4.4 and require one of the following three requirements:
  - Develop a ductile moment connection having a minimum drift angle of 0.02 radians. The determination of drift angles for various moment connections requires reference to test data and pre-qualified joints. See the references listed in Commentary J but start with the CISC publication *Moment Connections for Seismic Applications*.
  - Beam-column connections to develop a moment of at least $1.1R_yM_{pb}$ which is considered to be satisfied by welding the beam flanges and webs directly to the column flange using complete penetration groove welds with electrodes matching the column and beam steel. For 300W and 350W material would mean using weld material with an ultimate tensile strength of 490 MPa (E49XX electrodes). Note that both the beam web and the beam flanges are connected with complete penetration welds. A beam-column connection in which the flanges were welded with fillet welds or the beam web was connected with a bolted connection would not satisfy this requirement.
  - Develop the moments equal to the gravity load moment combined with the seismic load multiplied by 2.0 provided the "controlling limit state is ductile" which would bring you back to one of the recognized moment connections in the references of Commentary J including the CISC publication *Moment Connections for Seismic Applications*.
- For the beam flange to column flange connection, use complete penetration welds (no partial penetration welds or fillet welds).
- Top flange backing bars and run-off tabs may remain provided they are continuously welded to the column flange on the edge below the complete joint penetration groove weld.
- Remove bottom flange backing bar and weld run off tabs and repair with a fillet weld.
- In single-storey frames, the beams frame over columns. In all other cases, the beams frame into the side of the column.

Probably the most common way to determine the forces in moment frames prior to doing capacity design is to use a frame analysis program and compute the forces. Some of these programs will have programs that do automatic design of steel members but the user should exercise extreme caution in such cases, as the resulting designs will seldom meet seismic design requirements.

The major problems include the following:

- The programs will often introduce different columns in one lift as opposed to making the column continuous over the maximum length to eliminate costly column splices.
- The programs that do automatic design of steel structures will seldom check the column for the forces derived from yielding of the beam.
- Many moment frames will be drift controlled (making moment frames satisfy the drift requirements) will often produce member sizes that make it pointless to detail to requirements greater than Limited Ductility $R_d = 2.0$). Most automated design programs concentrate on adjusting member size to meet strength requirements not drift limitations. When calculating drift the elastic drifts should be multiplied by (R_dR_o/I_E) and should be increased for P-Delta effects by multiplying by the U_2 factor which is increased in accordance to 27.1.8 to account for seismic effects and limited to a maximum of 1.4.

Sizes resulting from analysis of this frame, including meeting the strength requirements and normal [illegible] tance drift limit requirements of 0.02hs (evaluated using an elastic structure with forces corresponding

$R_dR_o = 1$ and $I_E = 1$), are shown in Figure 16.7. The frame shown in Figure 16.7a meets strength and drift requirements. The column size of W360 has been chosen as many of the pre-qualified joints require this as the maximum column size.

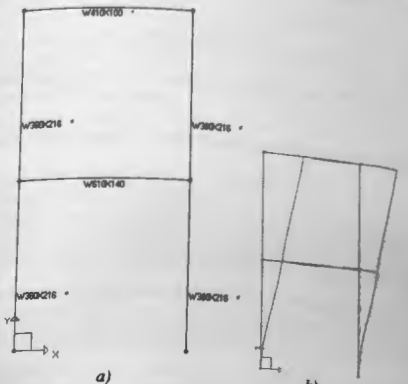


Figure 16.7. Sizes resulting from analysis of the frame

Deflected shape is shown in Figure 16.8.



Figure 16.8. a) Resulting shear. b) Moment diagram

Examining the results for the middle beam, we find that the beam has a factored moment of 1227kNm and a shear of 575kN. However we must push it to its plastic moment and examine the effect of these forces on the column.

W610x140 M_{pb} = Plastic Beam moment
M_{pb} = $F_y * Z$ (No Ø, No R_y for $R_d = 2.0$ frames)
= 345,000kN/m² * 4150 x 10^{-6} = 1432kNm

[illegible]city, Figure 16.9 shows only the seismic forces but the 1.0D + 0.5L that corresponds to this seismic [illegible] should also be included.

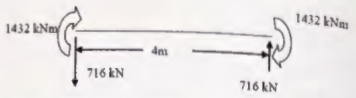


Figure 16.9. Isolating the beam with yield forces at each end

We now have to check that the column has the capacity to resist the beam moments. This is done using Clause 27.2.3.2 substituting M_{bp} for $1.1R_yM_{pb}$ in our $R_d = 2$ moment frame.

M_{pc} = Nominal Plastic moment resistance of column. (4260 * 345,000 = 1470kNm)

$$M'_{rc} = 1.18\phi M_{pc}\left(1-\frac{C_f}{\phi C_f}\right) < \phi M_{pc}$$

C_y = Axial capacity at yield stress: 27,600 * 345,000 = 9522kN

C_f = Axial load on column including the seismic shears from the beams when they reach their plastic moments.

In our case this comes to:

$$\sum M'_{rc} \geq 1431kNm + 715(0.380) = 1703kNm$$

The moment resistance where the beam meets the column is derived from the moment resistance of the column above the beam and the column below the beam.

$$M'_{rc} = 1414kNm < 0.9*1470kN = 1323kNm$$

$$\sum M'_{rc} = 2*1323kNm = 2646kNm$$

This must be compared to the moment that the beam is exerting:

In our case assuming the hinge is 200mm from the column face we get:

$$\sum M'_{rc} \geq \sum\left(1431+715(0.2+\frac{.360}{2})\right)$$

$$\sum M'_{rc} \geq \sum\left(M_{pb}+V_h(\alpha+\frac{d_c}{2})\right)$$

$$\sum M'_{rc} \geq 1431kNm + 715(0.380) = 1703kNm$$

Our column resistance to moment is 2646kNm which exceeds the exerted moment of 1703kNm and therefore produces a satisfactory result.

We would now go on to selecting a ductile moment connection (stiffened end plate, dog-bone) and go through the detailing requirements for that system and look at the need for horizontal stiffeners and doubler plates in the column.

The following should be noted about the foregoing discussion:

a) The Code now requires that we not only bring the beam up to yield but we have to design for $1.1R_yM_{pb}$ where M_{pb} is F_yZ. For $R_d = 2.0$ moment frames we can reduce from $1.1R_yM_{pb}$ to M_{bp}.

b) Columns where there are beams framing in on both sides will have more difficulty in satisfying the strong column weak beam requirement especially if column size is kept to less than W360 as required by several of the pre-qualified connections.

c) Frames where the beam spans are longer or the dead and live loads on the beams are greater will pose more challenges as the beam size will increase to account for these factors and the column size will have to follow.

d) When using reduced beam sections the maximum moment that the beam can exert on the column at the column face is less than $1.1R_yM_{pb}$ and this can be taken advantage of in the design of the column.

e) Under some circumstances the Code permits the use of hinges at the tops of columns in single-storey frames. In these conditions the hinging column fastens to the underside of the beam and the roles of beam and column are reversed in the approved beam-column joint assemblies.

f) There is no such thing as a soft storey in a single-storey frame and some of these restrictions such as having strong column weak beam can be reduced for single-storey buildings.

16.5 REQUIREMENTS FOR JOINTS IN MOMENT FRAMES USING REDUCED BEAM SECTIONS OR "DOGBONES"

The requirements of the pre-qualified joints that use the reduced beam section are defined in FEMA documents such as FEMA 350, the CISC manual *Moment Connections for Seismic Applications* or the actual test data report for the certified joint. The designer who is proposing to use reduced beam sections will encounter the following issues:

1) The section capacity at the reduced beam section should be sufficient to satisfy the factored loads computed for the centre of the reduced beam section, using the Z for the section at the reduced beam section and a stress of Phi times the yield stress. Load combinations for factored dead and live and factored load cases involving wind and seismic loading should be satisfied. Some of the certified joints want much of the load to be seismic as opposed to dead and live load.
2) The stiffness of the frame, including the effect of RBS sections, should be sufficient to meet the drift requirements under seismic loading. To evaluate the effect of RBS on drift, use a model with a joint at each end of the RBS and put the inertia for the RBS in the segment between the joints.
3) The stress in the weld where the beam flange is welded to the column flange is to be evaluated using a plastic modulus, Z, for only the flanges of the beam. The moment that is to be evaluated at the beam-column intersection is computed with the probable moment at the RBS at both ends of the beam (use 1.26 times the yield of the beam when evaluating the probable moment). The moment at the beam-column weld is the moment at the centre of the reduced beam section plus the shear at the reduced beam section times the distance between the centre of the reduced beam section and the weld. The maximum stress permitted for full penetration welds is F_y of the base material.

(The 1.26 in the above comes from $C_{pr}R_y$ where $C_{pr} = (F_y + F_u) / (2F_y)$

4) There may be specific requirements on limiting the through thickness stress for the column flange under gravity and seismic forces sufficient to develop the probable moment at the RBS at each end of the beam. Note this item has similar effect as item (3) however, the purpose of item (3) is to protect the beam flange to column flange weld while item (4) is to protect from though thickness fracture of the column flange.
5) The plastic section modulus at the reduced beam section may be calculated as

$$Z_{RBS} = Z_x - b_R * t_f * (d - t_f)$$

where: Z_{RBS} = Plastic section modulus of the reduced beam section
Z_x = Plastic section modulus of the unreduced beam section
b_R = total width of material removed from the beam flange
t_f = the thickness of the beam flange
d = the depth of the beam

6) The probable plastic moment at the M_{PR} at the centre of the reduced beam section shall be calculated from the equation:
$M_{PR} = Z_{RBS} * (1.26)F_y$. The 1.26 factor depends on the mean yield stress on the beam material and the estimated strain hardening and 1.26 applies to ASTM A572 and A992 steel. Some of the pre-qualified joints may require that A992 steel be used in the reduced beam section which would overrule the use of G40.21.
7) Check to see that strong-column-weak-beam criteria is met. The moments in the column when the probable moment in the RBSs are reached should be resisted by the column section at $Z_c*(F_y - f_a)$, where f_a is axial stress in the column and Z_c is the plastic section modulus for the column.
8) All corners of the RBS should be rounded to minimize notch effects, all cut edges should be ground in the direction of the flange length until they have a surface roughness value of less than or equal to 1,000.
9) The RBS should reduce 30% to 40% of the beam flange though at times this can go as high as 50% of the beam flange. If there are specific requirements on the pre-qualified joint these should be used.
10) The start of the reduced beam section (dimension "c" in Figure 16.10) should be approximately d/4 where "d" is the beam depth. Again there may be specific requirements on the pre-qualified joint these should be used.

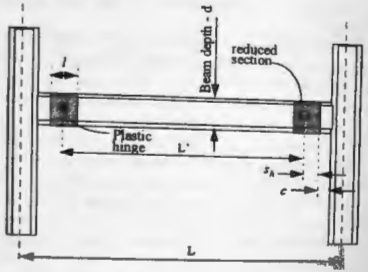


Figure 16.10. General configuration of the reduced beam section

11) The length of the reduced beam flange (Dimension "l") in Figure 16.10 should be 0.75d to d. See the pre-qualified joint requirements.
12) The shear capacity of the beam and the bolted connection for transfer of shear to the column should be capable of supporting the loads occurring in the beam span as well as the hinging induced shear. The hinging induced shear is computed from the probable ($1.26F_y$) plastic moment occurring at the RBS at both ends of the beam.
13) Both flanges of the beams may be required to be provided with restraint on the non-column side of the RBS to prevent lateral torsional buckling. Attach the top flange to the decking. Attach lateral braces to the bottom flange. The capacity of the brace shall be sufficient to develop 6% of the probable ($1.26F_y$) compression force in the flange at centre of the reduced beam section. This requirement varies depending on the pre-qualified joint -- the tests of deep-column moment frames from Lehigh found that the lack of restraint was beneficial to the post yield behaviour of

the system. Early FEMA documents leaned required the brace -- subsequent ones felt that a brace to the bottom flange was not required if the top flange was attached to deck.
14) The region of the reduced beam section is expected to have high plastic deformation demands under seismic loading and holes or welding attachments in this region are to be avoided as they can lead to stress raisers and premature fracture.

Design the actual weld connection in accordance to good post Northridge earthquake procedures:
- Complete penetration welds
- Remove backing bars on bottom flange weld
- Use non-destructive testing on every beam flange to column flange weld
- Use continuity plates and web doubler plates as required by calculation

16.6 END PLATE MOMENT CONNECTIONS

One solution to achieving a ductile moment connection is to use end plate moment connections as shown in Figure 16.11. This applies to R_d of 2.0 moment frames and can under some circumstances extended to R_d of 3.0 frames. The rules for these frames can be found in the CISC manual *Moment Connections for Seismic Applications*. As with other seismic systems it is required to follow all the detailing rules. The end-plate moment connection has the advantage that most of the fabrication occurs in the shop with no field welding required. Erection of the structure is very quick though at times some of the bolts required can require specialized equipment to install.



Figure 16.11. Example of an end plate moment connection

16.7 HORIZONTAL STIFFENERS AND WEB DOUBLER PLATES

Two very important details with moment connections are the horizontal stiffeners and the web doubler plates. The forces from the beam moment result in large point loads from the beam flanges acting on the column flanges. Some of this gets transferred directly to the column web while some of this load tries to bend the column flange. The column flange may need horizontal stiffeners to improve its capacity to resist these point loads. In the joint area the beam flanges are in opposing directions-there is one flange in compression while the other flange will be in tension. The tension force is equilibrated by the compression force through shear in the column web. The force can be quite large (as they are $A_{flange} * F_y$) and it may

be that the shear capacity of the column web is not sufficient, resulting in the need for a vertical doubler plate to increase the shear resistance of the column web. These principles are illustrated in Figure 16.12, which shows the horizontal stiffeners and doubler plates, and in a sample calculation.

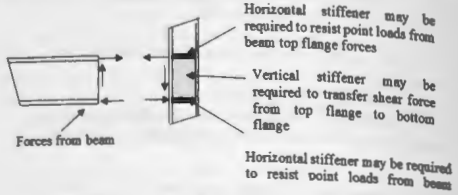


Figure 16.12. Forces for sample moment frame

EXAMPLE—DESIGN A HORIZONTAL STIFFENER IN A COLUMN AT A MOMENT CONNECTION

The requirement for horizontal stiffeners is due to the tensile or compressive force from the beam flange acting as a point load on the column. Under capacity design the load from the beam flange should be its yield load (Area of Flange * Yield stress for beam). The resistance of the column flange to these forces can be determined using the requirements of S16-09 Clause 21.3. Horizontal stiffeners are required when these equations are exceeded.

The bearing capacity opposite the compression flange: $B_r = \phi_b w_c (t_b + 10t_c) F_{yc} < \frac{M_f}{d_b}$

The tension capacity opposite the tension flange: $T_r = 0.6 * 7\phi (t_c)^2 F_{yc} < \frac{M_f}{d_b}$

The terms are defined in Clause 21.3 but the 0.6 factor in the front of the T_r equation comes from Clause 27.4.4.3.

Under post-Northridge Earthquake design the weld between the flange of the beam and the column should always be a complete penetration weld but the force transmitted to the horizontal stiffener is reduced by the flange of the column and the weld between the horizontal stiffener and the inside face of the column can be a fillet weld.

The capacity of web to resist shears from the flange forces is given in S16 Clause 27.2.4.2. The shear is best determined by drawing a free body diagram of the joint. In the past the yielding of the panel zone has been used as a seismic yielding element to act as a fuse to limit the seismic forces. Since the Northridge Earthquake this method of energy dissipation has somewhat lost favour. Yielding of the panel zone may be disrupted by stiffeners and beam supports. The capacity of the panel zone is given as:

$$V_r = 0.55\phi d_c w' F_{yc} \left[1 + \frac{3b_c t_c^2}{d_c d_b w'}\right] < 0.66\phi d_c w' F_{yc}$$

The subscript c refers to column while the b subscript refers to beam. If the shear capacity of the panel zone is exceeded the capacity can be increased by adding doubler plates or inclined stiffener plates.

If the analysis of beam-column intersections is to be undertaken frequently it is suggested that many of the tasks be computerized.

Figure 16.13 shows two reduced beam sections connecting to a cruciform column providing stiffness frames in two directions.



Figure 16.13. Beam-column intersection providing stiffness in two directions

CHAPTER 17
SEISMIC DESIGN OF ECCENTRICALLY BRACED FRAMES

Eccentrically braced frames are an effective system for handling high seismic forces as they take advantage of high ductility and R_d values to reduce your forces. Of all the seismic systems currently in S16 the EBF provides the highest R_d values that can be achieved in practice. Figure 17.1 shows the construction of a post-disaster building in Vancouver that was located on soil where the site specific spectrum gave very high lateral loads that could only be accommodated with using a very ductile system.



Figure 17.1. Eccentrically braced frames being used in a post-disaster building under construction in Vancouver

The configuration of the EBF can often get confused with the chevron brace but unlike the chevron brace the brace elements of the EBF deliberately do not meet at a single point. The eccentricity of the braces not

only gives the EBF its name but it provides for a region of beam that can yield in either shear or moment. The designer decides if the yielding mechanism is to be shear or moment and the link element is detailed accordingly. There are extensive rules in S16 to make sure that the yielding does occur in a sustained manner. Figure 17.2 shows the link element in an EBF that is designed to yield in shear. The link has several plates that have been added to improve the shear resistance and have the link behave in a ductile manner.



Figure 17.2. Link Element in an eccentrically braced frame

The EBF is a much more ductile system than the concentrically braced system as the yielding in shear or moment of the link element is much more ductile than the axial yielding of the braces which occurs in the concentrically braced frame system.

17.1 DUCTILE ECCENTRICALLY BRACED FRAMES – CLAUSE 27.7 (R_d = 4.0 R_o = 1.5)

Eccentrically braced frames are a hybrid system in that the frame has brace members but the yielding occurs in the link beam where a gap exists between the beam-brace intersections. The yielding may occur in shear or bending and this acts as a fuse to control the forces acting on the structure. This system has proved to be quite effective in resisting seismic loads and displays good ductility. The requirements for this bracing system are given in Clause 27.7. The possible location of yielding elements of eccentrically braced frames are shown in Figure 17.3.

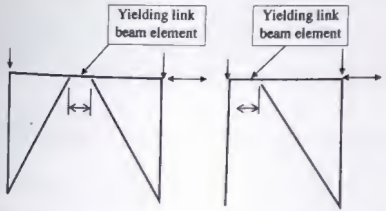


Figure 17.3. Yielding elements of eccentrically braced frames

The link beam has various requirements that must be satisfied, as follows:

- Links are provided at one end of every brace. The link is designed as the yielding element and may yield either in moment or shear.
- The links are Class 1 elements and generally I-shaped though the use of HSS links has also been tested.
- The code provides special resistance formulas for the link elements, depending on yielding in shear or moment.
- The length of the link element must exceed its depth.
- The link must have capacity for elastic and inelastic rotations.
- The brace bay must have braces of sufficient capacity to develop the plastic yielding in the link element. The yielding is intended to occur in the link element not in the braces. The braces are intended to remain elastic throughout the seismic event.
- The link must have braces to both top and bottom flange at each end.

17.2 ADVANCES IN ECCENTRICALLY BRACED FRAMES

Research at the University of New York at Buffalo has resulted in the concept of a "fusible link" in the EBF in which an HSS link element is used between plates that are bolted in place. The desire is that after a seismic event the damaged link could be un-bolted and a new link placed in the EBF.

CHAPTER 18
SEISMIC DESIGN OF STEEL PLATE SHEAR WALLS

Steel plate shear walls are a ductile and versatile method of providing lateral resistance for a building but they are seldom used in Canada. Considerable research has gone on in North America to research and advance the system but they are not typically found in buildings. One example of their use is the control tower and office building at Edmonton Airport. Figure 18.1 shows the building under construction with the steel plate shear wall exposed and a detail of the shear wall.



a) b)

Figure 18.1. Steel plate shear wall for control tower and office building in Edmonton

18.1 STEEL PLATE SHEAR WALLS

Steel plate shear walls consist of a steel plate for carrying the shear centred between two vertical column elements that act to carry the moment. Horizontal beams stiffen the plates and careful attention is paid

to making sure that the plate does not tear away at the connections. The steel plate shear wall system becomes a vertical girder for carrying lateral loads. Steel plate shear walls can be detailed to two levels of ductility. (Type LD Limited Ductility $R_d = 2$ $R_o = 1.5$ Covered in Clause 27.10 and Type D ductile with $R_d = 5$ $R_o = 1.6$ which are covered in Clause 27.9) The method of energy dissipation is the yielding of the thin web plates with tension fields that develop as they carry shear. The yielding method is tension yielding of the web plate under shear loads. This sets up diagonal tension fields that yield under seismic loading. The thickness of the plate is selected such that shear yielding will occur. Like brace bays it is difficult to design steel plate shear walls for low shears, as the required thickness of the steel plates is too low, the commentary to S16 shows a sample structural model that can be used for determining the thickness of the web. Very thin webs (say 0.91mm or 1.22mm) would be subject to the same issues with connection that we have with deck diaphragms where we are trying to connect similar thickness of sheet steel to thicker elements.

Steel plate shear walls have been used successfully in California, an example being the Olive View Hospital in the San Fernando Valley. The hospital was heavily damaged six weeks after it opened in the magnitude 6.6 San Fernando earthquake of February 9, 1971. The failures observed at the Olive View Hospital in 1971 included seismic resisting stair cores falling away from the building and soft storey failures influenced subsequent seismic codes throughout North America. The building was demolished and a replacement opened in 1987 with a seismic system from steel plate shear walls. The rebuilt hospital fared well following the 1994 magnitude 6.8 Northridge earthquake which affected the same region as the 1971 event.

Research has shown that it is possible to still have ductile behaviour in steel plate shear walls with circular holes cut in the plate and with the corners cut out of the steel plate at the beam-column intersection. Based on this research S16 now includes provisions for both these types of shear walls.

Steel plate shear walls have rigid frames that surround the steel plate with the beams and columns moment connected. Many of the detailing rules for the shear wall concern the detailing of the beams and columns that surround the shear wall. The intent is that the columns remain elastic without shear yielding of the beam-column panel zone and with yielding occurring in the surrounding frame only at the base of the columns.

18.2 STEEL PLATE SHEAR WALL EXAMPLE

As example of a steel plate shear wall let's take the two-storey frame that we have used with other systems and determine the thickness required for the steel plate of the shear wall. The geometry remains unchanged but we will change the forces to match the $R_d = 2.0$ $R_o = 1.5$ values of the steel plate shear wall. The forces given in Figure 18.2 include for eccentricity effects and notional loads.

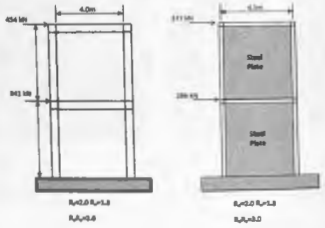


Figure 18.2. Sample steel plate shearwall with forces shown

Our first desire is to determine what the thickness of the steel plate should be.

S16 gives the resistance of the steel plate wall as:

$$V_r = 0.4\phi F_y w L \sin(2\alpha)$$

where: w = thickness of plate – what we are looking for.
L = centre to centre distance of columns in a plate wall (4.5m in our case)
α = angle of inclination of the tension field of the plate wall from vertical and is computed in accordance with S16 clause 20 which states that α may be taken as 40 degrees if the aspect ratio of the panel satisfies:

$$0.6 \le \frac{L}{h} \le 2.5$$

Where h = storey height

In our case L = 4.5m and h = 4.5m we therefore have an aspect ratio 0f 4.5m / 4.5m = 1.0 and we are in the acceptable range for aspect ratio to use $\alpha = 40$ degrees.

Rearranging our formula for V_r to solve for w we get:

$$w = \frac{V_r}{0.4\phi F_y L \sin(2\alpha)} = \frac{673kN}{0.4 * 0.9 * 350{,}000\frac{kN}{m^2} * 4.5m * \sin(2 * 40°)} = 0.00178m = 1.78mm$$

This would result in sheet steel of 1.22mm thickness commonly used for decking being used which would have a yield of 240MPa requiring a recalculation and a resulting thickness of 1.78mm of 240MPa material which would get bumped to 1.95mm (14ga) material. This is still quite a thin section and would have the same issues with fastening as does a sheet of similar thickness deck material. A thin plate for the infill plate would also give rise to concerns that the sheet would be subject to damage or oil-canning in use. Once the plate thickness has been selected the frame surrounding the plate can be designed and detailed.

CHAPTER 19
CONVENTIONAL CONSTRUCTION AND LOW SEISMIC DESIGN

S16 has a seismic clause labelled "conventional construction" which is intended for use on buildings that for a variety of reasons do not fit into the more ductile clauses of the standard. The conventional construction clause recognizes that steel in itself is a ductile material and given a careful designer a steel structure will try to behave in a ductile manner before fracturing. There are deceptively few requirements listed in Clause 27.11 – indeed if we are not dealing with a cantilevered construction situation (which we will deal with in Chapter 19) and we are dealing with a structure that is less than 15m high we only have a single clause that needs to be satisfied:

27.11 Conventional construction, $R_d = 1.5, R_o = 1.3$

27.11.1

Structural systems in this category have some capacity to dissipate energy through localized yielding and friction that inherently exists in traditional design and construction practices. Except as otherwise specified in Clause 27.11, the requirements of Clauses 27.1 to 27.10 and 27.12 shall not apply to these systems.

Diaphragms and connections of primary framing members and diaphragms of the seismic-load-resisting system of steel-framed buildings with specified short-period spectral acceleration ratios ($I_E F_a S_a(0.2)$) greater that 0.45 designed to resist seismic loads based on a force reduction factor, R_d, of 1.5 shall be

a) Proportioned so that the expected connection failure mode is ductile; or
b) Designed to resist gravity loads combined with the seismic load multiplied by R_d.

The connection design load need not exceed the gross section strength of the members being joined, as determined using the probable yield stress $R_y F_y$.

This is misleadingly simple and it might be tempting to consider that we should design all of our buildings this way, with no troublesome capacity design to worry about and we just have to consider that the connections to have a ductile failure mode and even that can be ignored if we want to design for large enough forces. The "conventional construction" clause attempts to reduce the complex seismic behaviour to that of a "static" lateral load such as wind.

To qualify for the forces that correspond to $R_d = 1.5$, $R_o = 1.3$ the designer must design the connections as having a ductile failure mode(S16 Commentary):

- Extended end plate moment connections (see Figure 19.1a)
- Flange plate moment connections (see Figure 19.1b)
- Gusset plates proportioned for ductility

- Welded connections loaded primarily in shear
- Bolted connections where the governing failure mode is bolt bearing failure

To achieve these ductile failure modes it would be necessary for the structural engineer to detail these connections to the extent that their ductility could be assured.

If it is not possible to assure ductility in the connection, then the structural engineer is required to design the connections of seismic elements for their gravity load plus the seismic load multiplied by R_d. We would therefore be connecting for the $R_d R_o = 1.3$ loads but would escape much of the detailing requirements that come with using a seismic system with a higher R-value.

Keep in mind when designing using $R_d = 1.5$ systems that it is still your responsibility to design a viable seismic system following the forces through from diaphragm to foundation and doing the best to provide the most ductility that can be achieved. It is also worth noting that the Building Code puts restriction on $R_d = 1.5$ structures including requiring that they be a maximum of 15m high in any location where the short period spectral acceleration ratio ($I_E F_a S_a(0.2)$) is greater than 0.35. The National Building Code does not permit $R_d = 1.5$ structures for post-disaster buildings.

Examples of moment frames beam / column joints that meet the requirements of ductile connections are shown in Figure 19.1.

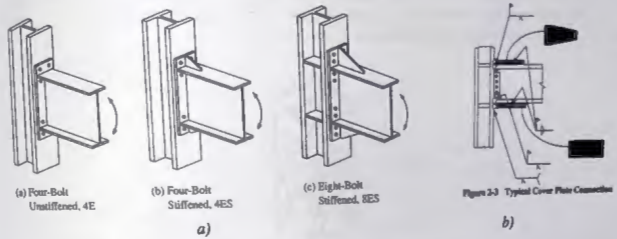


Figure 19.1. Examples of types of ductile moment connections: a) Stiffened end plate connections. b) Flange plate moment connections
Source: a) AISC manual; b) FEMA

The structures that seem to gravitate toward "Conventional Constructioa":

1) Buildings that do not have a well-defined yielding location. Such an example would be the "Bird's nest" stadium in Beijing, where large members will not provide a yielding location. Buildings with wild architectural layouts that do not follow the standard layout of rectangular bays often fall into this category.
2) Buildings whose configuration violates $R_d = 2$ or greater requirements. For example, the frame at the right is a part of a single-storey industrial building and has wide-flange columns at each side and a roof truss that is connected to the column at top and bottom chords, as shown in Figure 19.2. This would appear to be a valid moment frame but it violates the requirements for even $R_d = 2$ moment frames by not having a Class 1 or 2 beam and not having moment frame recognized beam-column connections.

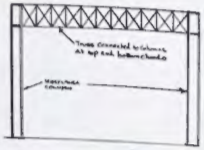


Figure 19.2. Truss connected to columns

3) Buildings with expansions in which the original building, because if its vintage, does not satisfy $R_d = 2$ or greater detailing and it is desired to expand the building without an expansion joint but have the co-joined building comply with the Building Code. The solution to this might be to have the new building satisfy $R_d = 1.5$ detailing.
4) Buildings with member sizes that are not recognized in the present code such as rod bracing or beyond the limits of testing such as a frame that contains W1200 columns when ductile behaviour and certified moment connections are only available up to W900 columns.

Buildings which have hybrid systems such as the small structure shown in Figure 19.3, where braces at the lower floor provide stability for a moment frame with HSS columns. Moment frames with HSS columns are not recognized as being viable in $R_d = 2$ or greater systems but this system would not be an acceptable $R_d = 2$ system even if wide-flange columns had been used.



Figure 19.3. 3-D view of hybrid column system

Even within structures with awkward seismic systems, the designer has a responsibility to work toward structural systems that have defined load paths and incorporate seismic details and provisions so that the building behaves in a reasonable manner under the highly variable and dynamic seismic loads. When designing new buildings in higher seismic zones, designers should strive to use recognized seismic systems with ductility factors greater than R_d of 1.5.

In NBCC 1995, the height limits for buildings were in terms of number of storeys and $R_d = 1.5$ buildings were allowed up to 3 storeys in higher seismic zones. This accommodated most industrial buildings and many designers used the "Conventional Construction" procedures to address challenging architectural buildings.

Under NBCC 2005 the height limits went from number of storeys to building height, which forced a variety of structures that previously could comply with $R_d = 1.5$ to fall outside the limits. The Structural Commentaries to NBCC 2005 (Commentary J item 143) tried to rectify this with the following statement: "The use of conventional construction for steel buildings in regions of moderate and high seismicity is restricted to buildings not exceeding 15m in height. This restriction is simply intended to retain the

traditional three-storey height limit stipulated in previous editions of the NBC; however this height limit was not intended for and does not apply to single-storey, steel industrial structures. In particular, structures such as steel mills and aircraft hangers may exceed 15m in height and are permitted to be built using conventional construction ..." What was happening was that the commentary to NBCC 2005 was saying the opposite of the Code but was intended to apply to industrial buildings with lower occupancies. Commentary item 143 goes on to state: "Other tall steel structures such as stadia, exhibition halls, arenas, and convention centres must satisfy the height restrictions ..."

Under NBCC Clause 4.8.11.3(b) the building fundamental lateral period for brace bay buildings is given as:

$T_n = 0.025h_n$ where h_n is the height of the building in metres. For a 15m tall brace bay building this is:

$T_n = 0.025h_n = 0.025*15 = 0.375$ seconds.

If we use the static analysis procedures, we see that for Vancouver the R_d of 1.5 structures will be restricted to the area of the 2/3 cut-off plateau for seismic force and would be designed for 0.26g to 0.35g depending on soil type is as shown in Figure 19.4.

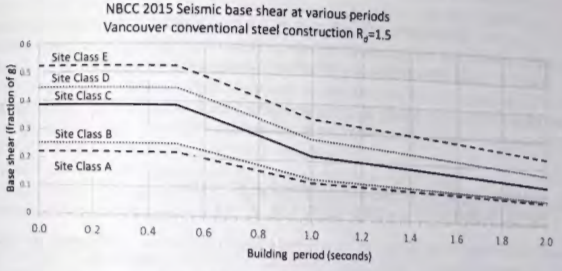


Figure 19.4. Brace bay type building

Figure 19.4 shows the base shear for an $R_d = 1.5$ brace bay type building in Vancouver. Period for a 15m structure of 0.375 seconds is shown. Regardless of soil type the building period is still on the 2/3 cut-off in base shear.

If the connections were not ductile (as defined earlier) the connections would need to be designed for the forces multiplied by $R_d = 1.5$ giving us forces of 0.39g to 0.53g.

What this discussion is trying to indicate is that the seismic forces on "conventional construction" buildings can get large. What the Code is trying to do is force the strength of the seismic system to increase to the point where even under the design level seismic event there are few demands on the building's seismic system to undergo excursions into the inelastic range.

19.1 CONVENTIONAL CONSTRUCTION UNDER S16 AND NBCC 2015

With the height limit of 15m in high seismic zones it was felt that it was necessary to extend the provisions to greater heights particularly for industrial structures. The desire being to address the contradictory information in the Structural Commentaries that tried to eliminate the height limits for this type of structure. The rules for using a "conventional construction seismic system" for buildings over 15m are onerous and will drive many designers to use a more ductile system.

Under S16 the rules for conventional construction steel frames over 15m are:

1) The structure must be normal importance and not assembly occupancy (this eliminates schools, stadiums and other buildings where people will gather).
2) The factored seismic forces increase by 2% per meter over 15m. So by 40m the design force has gone up by 50%.
3) The height limit is raised to 40m in high seismic zones and 60m in medium seismic zones.
4) Dynamic analysis procedures must be used to get the distribution of forces in the structure.
5) Requirements of Clause 27.1.3 to 27.1.8 must be satisfied. This includes requirements on the steel used.
6) All members in the seismic force resisting system must be Class 1 or Class 2.
7) The width to thickness ratios for elements in the seismic force resisting system must correspond to those appropriate for systems with an R_d of 2.0
8) Columns must be designed to resist the 1.3 times the compression forces calculated from seismic action and if the column is located at an intersection of two frames must resist 100% of the forces in one direction and 30% of the forces in the other direction.
9) Connections are designed for gravity load plus 1.3 times the member factored seismic loads and must have a ductile failure mode.
10) Diaphragms must be designed for $R_dR_o = 1.3$ forces – it is not possible to use "ductile diaphragms" to limit the forces.
11) For columns with members of the seismic force resisting system that intersect the column at a point in a column that does not correspond to a floor level must be designed for an out of plane load at that point corresponding to 10% of the factored axial load of the column.

It is worthwhile emphasising that R_d of 1.5 Conventional Construction detailing is only acceptable in assembly occupancy buildings of less than 15m in high seismic risk areas.

19.2 CANTILEVERED COLUMN CONSTRUCTION: REQUIREMENTS UNDER CLAUSE 27.11

The most readily recognizable example of cantilevered column construction is the support for the canopies over the gas pumps at some gas stations, such as is shown Figure 19.5. In this method of support, columns that carry the axial load cantilever from a foundation act as the seismic system. Cantilevered construction is often mistakenly considered to be a moment frame and given the same R_d-value as a moment frame but this system is a poor seismic system and should be approached with caution.



Figure 19.5. A cantilever column structure under construction. In this case both the resistance in the longitudinal and transverse directions are provided by the column cantilevering from the foundation

Problems with cantilevered construction:

1) The columns are often slender and may have problems with lateral buckling before they can dissipate energy through plastic yielding of the base. Unless detailed properly the columns may have high b/t ratios that result in local buckling prior to the attainment of a plastic moment at the base.
2) The columns in a cantilevered column system usually are carrying the entire vertical load in the system and will be subject to P-delta effects. This situation is made worse by the dynamic interaction of the roof beams and the column as the roof beams can impose a moment on the top of the column which we are ignoring by considering the roof as a lumped mass at the top of the column.
3) The columns may not have a second frame present to assist in restore them to their original position once the seismic event has concluded.
4) The foundations and anchor rods are often not designed to resist loads that can be expected at time of hinging.
5) The location of hinging occurs at the connection point between the foundation and the column.

The American Uniform Building Code defines cantilevered seismic systems as being a separate category and gives this type of construction the lowest R-value of any system. Cantilevered column systems are rated even LOWER than masonry. The Uniform Building Code places severe limitations on this type of construction including limiting the structures to a maximum height of 35 feet (10.7m).

While the type of structure imagined for cantilevered construction may be considered to be the gas pump canopy, the system has been used on reasonably large buildings and the seismic behaviour of these buildings may not be satisfactory. Requirements for this type of structure were made more restrictive in S16-14.

Requirements under S16 impose the following restrictions on cantilevered column construction.

a) Use Class 1 columns
b) Use $R_d = 1.5$ $R_o = 1.3$ forces when sizing the columns.
c) Limit flexibility and P-Delta effects by requiring that U_2 be 1.25 or less.
d) Require that the base connections resist $1.1R_y$ times the nominal flexural resistance of the column but need not exceed R_dR_o=1.0 forces.

EXAMPLE–DESIGN COLUMNS FOR CANTILEVERED ROOF

A service station island roof to be located in an area where the base shear computed with $R_dR_o = 1.5 \times 1.3 = 1.95$ is .239W and is to be supported by two cantilevered columns as shown in Figure 19.6. The roof is 5m wide by 12m long. The roof is essentially flat and has a weight including steel structure of 1kPa. The centre of mass of the roof is 4m above the concrete base. Design the steel columns.

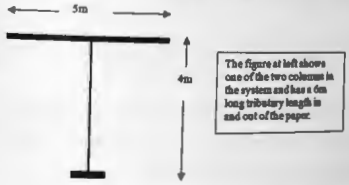


Figure 19.6. One of the two columns and a 6m long tributary length

Solution:

Determine the seismic loading and overturning moment. As a cantilever column structure we will [illegible] using $R_d = 1.5$, $R_o = 1.3$ forces, and the column must meet the requirements of a Class 1 section. Foundation should be considered but is ignored in this example for simplicity.

Seismic weight per column = (Dead load + 25% snow) * (Tributary area)

= (1kPa + 0.25*1.58kPa) * (5m * 6m) = 41.85kN

I_E = 1.0 though it might be argued that the "building" contains toxic material and should be high importance.

V = 0.239W = 0.239 * 41.85kN = 10.0kN

M = Height * V = 4m * 10.0kN = 40.0kNm

Axial load = Dead Load + 0.5*(Live load) or

= 1kPa (5*6) + 0.25*1.58kPa *(5m * 6m) = 41.9kN

Note that under current Code requirements we consider 0.25 times snow for our calculation of lateral loads AND the vertical load effect.

As a cantilevered structure we should use L_u of 2*4m = 8m when calculating axial resistance but as a square HSS with $I_x = I_y$ the unsupported length does not reduce the factored moment resistance.

Try 152x152x6.4 in G40.21 Class C: C_r = 305kN M_r = 61.7kNm (column capacity tables [green pages] of the CISC Handbook). The b/t ratio for this section is 26.3 and easily satisfies the 37.1 requirement for Class 1 (See section 5.5 of this text).

$$\text{Interaction} = \frac{41.9kN}{305kN} + \frac{40.0kNm}{61.7kNm} = 0.137 + 0.652 = 0.79 \quad (<1\ OK)$$

Determine deflection under R_dR_o = 1 forces

$$\Delta = \frac{PL^3}{3EI} = \frac{(R_dR_o = 1.5*1.3)*10kN*(4m)^3}{3*200*12.6} = 0.165m$$

Maximum permitted = $0.025h_s$ = 0.025*4m = 0.100m

Increase I to (0.165/0.100)*12.6 = 20.8

Try HSS 178x178x6.4, I = 20.6

$$\Delta = \frac{PL^3}{3EI} = \frac{(R_dR_o = 1.5*1.3)*10kN*(4m)^3}{3*200*20.6} = 0.101m$$

Examine the P-Delta effects using R_dR_o = 1 deflections:

Use Clause 8.6.2 $$U_2 = \frac{1}{1 - \left[\frac{\sum C_f \Delta_f}{\sum V_f h}\right]} = \frac{1}{1 - \frac{41.9kN*0.101m}{10.0kN*4m}} = \frac{1}{1 - 0.1058} = 1.12$$

(This is less than the maximum of 1.25 permitted for cantilever column structures.)

M_f = 40.0kNm + 1.12*41.9kN*0.101m = 40.0kNm + 4.73kNm = 44.7kNm

Deflection including P-Delta = 0.101m * 1.12 = 0.113m

Need I = (0.113/0.101) * 20.6 = 23.3 Use 178x178x8.0, I = 24.8

Check interaction with P-Delta effects included and using HSS 178x178x8.0
Check Class: b/t = 22.3 easily satisfies the requirement of 37.1 for Class 1.

C_r = 562kN at kL = 8.0m, M_r = 104kNm

$$\text{Interaction} = \frac{41.9kN}{562kN} + \frac{44.7kNm}{104kNm} = 0.075 + 0.430 = 0.50 \ (<1\ OK)$$

Examine the factored stresses in the column at this point:

$$\sigma = \frac{P}{A} + \frac{M}{S} = \frac{41.9kN}{5230X10^{-6}m^2} + \frac{44.7kNm}{279x10^{-6}m^3} = 8,000kN/m^2 + 160,000kN/m^2 = 168,000kN/m^2 = 168MPa$$

What has happened is that our R_d = 1.5 R_o = 1.3 forces do not come even close to yielding the column. Our design has become controlled by deflection requirements and by P-Delta effects. These ... to "tell" the structure to yield at the R = 1.5 forces and the force on the column would continue to rise under the design earthquake. Recall the force-displacement graph from Chapter 13 on capacity design, ... here as Figure 19.7.

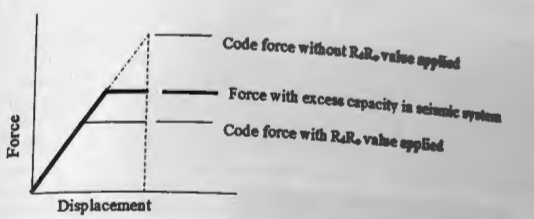


Figure 19.7. Force-displacement response

A detailed analysis that took account of any yielding would show that the seismic overturning moment will continue to rise as there is no limiting fuse. The code therefore requires that we connect the column for 1.1*R_y*Nominal Flexural capacity.

Design connection between column and footing for:

$M_f = 1.1(R_yF_y)Z = 1.1 * 460,000kN/m^2 * 330 \times 10^3 \times 10^{-9} = 167kNm$

(Noting in the above equation that we have used a value of 460MPa for the R_yF_y value of HSS sections.)

The value of M_f computed above is considerably greater than the "normal" section capacity of the column and vertical stiffeners will be required to make the connection work. A fillet weld between the column and the base plate will not develop the forces required.

Check R_dR_o=1.0 moment with P-Delta forces added = 40.0kNm*(R_dR_o=1.95)+4.73 kNm = 82.7 kNm (governs)

It would also be appropriate to design the foundation for the computed connection moment although it could also be argued that the limiting values of the concrete code would also apply and it would be possible to design the footing for the greater of R_dR_o = 2.0 moments or 75% nominal capacity of the SPRS if un-anchored or R_dR_o = 1.3 moments if tied down. Note that if the column had been a wide-flange column we must protect against lateral buckling of the column occurring prior to yielding.

19.3 OTHER SYSTEMS NOT CONSTRUCTED FROM STRUCTURAL STEEL

Systems not constructed from structural steel but often part of steel buildings include:

1) Reinforced concrete walls
2) Reinforced concrete moment frames (rare in our area)
3) Reinforced masonry walls
4) Tilt-up concrete walls
5) Strapped steel stud (This is more often used with load-bearing steel studs where there is little significant structural steel.)
6) It would be possible to have structural steel and plywood walls but this combination is not frequently employed.

The fundamental issue is developing sufficient connection between the steel and the non-structural steel seismic resisting system and detailing drag struts and connections to carry seismic forces. The required connections between the seismic system and drag struts to diaphragms should be designed and detailed by the structural engineer and clearly shown on structural drawings. The interface between different systems is a constant coordination challenge. Figure 19.8 shows an example of a steel building where all the seismic loads are carried by the concrete core. In this case the seismic system is detailed in accordance with the requirements of A23.3. The connections between the steel frame and the concrete building would be detailed by the structural engineer for the project they are not left to the steel detailer. These connections would include axial forces for the transfer of drag forces and angles to transfer the shear in the deck diaphragm to the concrete walls.



Figure 19.8. Steel building where all the seismic loads are carried by the concrete core

19.4 PARTS AND PORTIONS APPROACH TO SEISMIC DESIGN

For items that are small and not really part of the main building structure it is common to use the "Parts & Portions" section of the code (Building Code Clause 4.1.8.18) to determine the seismic forces. In this the seismic forces are equal to:

$$V_p = 0.3 * F_a * S_a(0.2)\, I_E * S_p * W_p$$

Where:

S_p is the seismic response for the part and is defined as follows: $S_p = C_p A_r A_x / R_p$

C_p = Element factor from the Building Code (1.0 for penthouses)

A_r = Amplification factor (2.5 for penthouses)

A_x = Height factor = $(1 + 2h_x/h_n)$ (For a penthouse at the top of the building $h_x = h_n$ so $A_x = 1 + 2^*(h_x/h_n) = 3.0$ (for elements at the top of the building).

R_p = Element response modification factor = 2.5 for penthouses.

W_p is the weight of the part.

F_a is F(0.2) (This is a "patch" in the 2015 Code rather than reformulate the Parts approach)

I_E is the importance factor and

$S_a(0.2)$ are the seismic input value that is given in the table in the Code.

The type of seismic resisting system does not influence the force computed from the parts and portions procedures, the forces computed would be the same for a moment-frame penthouse or a brace bay penthouse. The Building Code gives several cases and corresponding values for C_p, A_r and A_x for machinery, fixtures, and small parts added to a building as well as for attaching walls including tilt-up panels to the diaphragm for out of plane forces. From a structural steel standpoint the most important of category of parts and portions is "Towers, chimneys, smokestacks, and penthouses when connected to or forming part of a building." This approach is often used for calculating the seismic load on mechanical penthouses rather than including them as a separate floor in the building model when doing seismic analysis. When calculating the deflection of a system from forces calculated using the parts and portions procedures, assume the forces are $R_d = 1$ $R_o = 1$ and it is not necessary to multiply by R_dR_o to get the "true" deflection however dividing by I_E would be appropriate. Similarly the forces calculated on small parts of the building are not reduced for the R_dR_o value of the resisting system and the forces on connections and diaphragms are not increased for capacity design of the seismic resisting system of the penthouse. This might appear to contravene the philosophy of capacity design but as we will see the resulting forces on penthouses are high and the philosophy is in line with the diaphragm forces corresponding to $R_dR_o = 1.0$.

Figure 19.9 shows a small mechanical penthouse on a larger roof. Parts and portions analysis is intended to apply to this type of structure.



a) b)

Figure 19.9. Mechanical penthouses requiring parts and portions analysis

EXAMPLE—DETERMINE SEISMIC FORCE

A mechanical penthouse is to sit on the roof of a post-disaster hospital located in a high seismic risk area of western Canada where $S_a(0.2) =$ is 1.304 and PGA = 0.580, the site soil classification is Site D, the snow values are $S_s = 1.1$kPa, $S_r = 0.2$kPa. The seismic forces for the penthouse are to be found using the parts and portions procedures of the code. The penthouse roofing and deck weigh 1.0kPa and suspended from the beams is approximately 2.0kPa of mechanical equipment of which half is counted for seismic loading. The roof dimensions are 7.5m x 10m. Determine the seismic force.

Solution:
The seismic weight is:

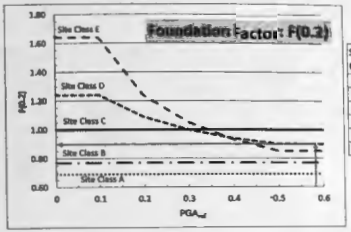
25% Snow = 0.25*(0.8*1.1 + 0.2)	= 0.27kPa
Roofing and Deck =	= 1.00kPa
Suspended Mechanical (50% of 2kPa)	= 1.00kPa
Beams (Don't use joist over mechanical rooms)	= 0.20kPa
Walls tributary	= 0.25kPa
	= 2.72kPa

Seismic Weight = 2.72kPa * 7.5m * 10m = 204kN

Seismic Shear $V_p = 0.3 * F_a * S_a(0.2)\, I_E * S_p * W_p$

This is a location in Western Canada so $PGA_{ref} = PGA = 0.580$

Using the procedures we learned for NBCC 2015 base shear calculations and Figure 19.10 we can determine F(0.2).



Site Class	PGA_{ref} ≤0.1	PGA_{ref} =0.2	PGA_{ref} =0.3	PGA_{ref} =0.4	PGA_{ref} ≥0.5
	0.1	0.2	0.3	0.4	0.5
A	0.69	0.69	0.69	0.69	0.69
B	0.77	0.77	0.77	0.77	0.77
C	1.00	1.00	1.00	1.00	1.00
D	1.24	1.09	1.00	0.94	0.90
E	1.64	1.24	1.05	0.93	0.85

Figure 19.10. Computation of F(0.2) for site class D where $PGA_{ref} = 0.580$

Site Class D where $PGA_{ref} = 0.580$ Get $F(0.2) = 0.90 = F_a$

$S_p = C_p A_r A_x / R_p = (C_p = 1.0) * (A_r = 2.5) * (A_x = 3.0) / (R_p = 2.5) = 3.0$

$V = 0.3 * (F_a = 0.9) * (S_a(0.2) = 1.304) * (I_E = 1.5) * (S_p = 3.0) * (W_p = 204kN)$

$V = 1.58\, W_p = 1.58 * 204kN = 323kN$

This is a very large number to design for – the diaphragm shear adjacent to the 7.5m side is 323kN / (2 sides * 7.5m) = 21.4kN/m which is pushing the capability of a deck diaphragm. A trussed diaphragm might be required in some cases along with substantial force resisting elements on each side. It should be realized that in doing this calculation we picked some of the worst conditions possible including being a penthouse, located on a post-disaster building in a very high seismic risk area.

19.5 SEISMIC DESIGN OF INDUSTRIAL STRUCTURES

Industrial structures have often created challenges for designers. These structures tend to differentiate themselves from out normal consideration of what a "building" looks like. Industrial structures differ from traditional buildings in the following ways:

- Industrial structures often do not have traditional "floors" or continuous diaphragms in the way that other buildings have. An industrial structure can consist of hoppers, tanks, or process equipment that is encapsulated in bracing
- Industrial buildings often have very strange mass distributions – an elevated tank would, for example, be a typical inverted pendulum. Adding in the effect of fluid sloshing that would occur if the tank was partly full makes the situation even more complex.
- While industrial structures may contain large amounts of hazardous materials that can be detrimental if spilled they usually have very few human occupants.
- Most industrial structures have exposed structure which leads to very low damping as there are no architectural components to cause damping.
- In industrial structures the process often takes precedence over the structural system. The structure of the building is designed to fit around the process equipment, which frequently leads to brace bays not being uniform and a series of irregularities.

There have been two approaches to addressing the seismic design of industrial structures in S16. The first of these was by S16-09 to introduce increased height limits for conventional construction these increased height limits only apply to buildings that were not assembly occupancy. In using these requirements S16 requires that the forces used for design be increased as the height of the structure increases.

The second approach to designing industrial structures was the introduction in S16-14 of Annex M (Seismic Design of Industrial Structures). This annex gives guidance to industrial structure designers on seismic issues but also provides some restrictions such as requiring redundancy in the seismic system. An industrial structure designed in accordance with Annex M would not be allowed to have a seismic system where the failure of a single brace would cause loss of the seismic system. A table of height limits for various types of industrial structures in ranges of seismic zones is also given.

19.6 SEISMIC DESIGN IN LOW HAZARD AREAS

Some parts of Canada are regarded as quite stable from a seismic point of view. Much of the Prairies and the Canadian Shield falls into this region, where seismic events are widely spaced both in time and by geography. Under NBCC 2010 and earlier, sites on firm ground were typically not required to have seismic design if $F_a S_a(0.2)$ was less than 0.12 with F_a being the foundation factor, which for rock and firm ground was 1.1 or less. With much of the Prairies having an $S_a(0.2) = 0.095$, this clause meant that even post-disaster facilities did not have to be designed for seismic loads if the building was located on reasonable soil in this stable region. Unfortunately there is a chance of having a seismic event in these stable areas, as illustrated by the 1989 magnitude 5.6 earthquake in Newcastle, Australia, that killed 13 people, injured 160, and caused about A$4billion in damage. Similarly, the magnitude 5.5 event in Saskatchewan in 1909 may not have caused much damage in the sparsely populated region but it was felt over a wide area. While the probability of an earthquake near one particular location is small, the probability that a potentially damaging event will occur in the stable region is real. The NBCC 2015 Code, which aims to protect life, now requires that seismic design be performed in all regions of Canada – even those situated in the stable region. The innovation in the Code is that for these stable areas the calculation of the seismic forces is easy and the resulting seismic design is similar to wind design. Low hazard seismic design applies when $I_E F_a S_a(0.2) < 0.16$ or $I_E F_a S_a(2.0)$ is less than 0.03, as follows:

F_a = 1.0 for Rock and stiff soil with N_{60} blow counts greater than 50

F_a = 1.6 for firm soil with N_{60} blow counts between 15 and 50

F_a = 2.0 for soft soil with N_{60} blow counts less than 15

The resulting seismic shear is $V_s = F_a S_a(T_a) I_E W_t / R_s$

R_s is 1.5 except where there is a weak storey (storey strength in one storey is less than in the storey above) or unreinforced masonry and in these two cases R_s = 1.0.

EXAMPLE—DETERMINE BASE SHEAR ON A BUILDING IN WINNIPEG

Determine the base shear (%g) for normal importance building located on a soft site (N_{60} blow counts less than 15) in Winnipeg. The building is a three-storey building with a period of 0.3 seconds.

The seismic properties for Winnipeg are shown in Table 19.1.

Table 19.1. Seismic properties for Winnipeg

Location	$S_a(0.2)$	$S_a(0.5)$	$S_a(1.0)$	$S_a(2.0)$	$S_a(5.0)$	$S_a(10.0)$	PGA	PGV (m/s)
Winnipeg	0.054	0.032	0.016	0.007	0.0013	0.0007	0.032	0.021

Determine if low seismic risk procedures can be used:

$I_E F_a S_a(0.2) = 1.0 * 2.0 * 0.054 = 0.108$ (this is less than 0.16 so meets criteria for low seismic risk)

$I_E F_a S_a(2.0) = 1.0 * 2.0 * 0.007 = 0.014$ (this is less than 0.03 so also meets the criteria)

Low seismic risk provisions can be used.

$S_a(T)$ is the S_a value for the period of the structure for our period of 0.3 seconds we do linear interpolation between $S_a(0.2) = 0.054$ and $S_a(0.5) = 0.032$ and get $S_a(0.3) = 0.047$

$V_s = F_a S_a(T_a) I_E W_t / R_s = (Fs = 1.5) * (S_a(0.3\text{sec}) = 0.047) * (I_E = 1.0) * Wt / (Rs = 1.5) = 0.047Wt$

As this is steel building design, we are also required to consider notional loads that for most seismic cases will be 0.5% of the dead load of the structure so the actual shear will increase to 0.047Wt + 0.005Wt = 0.052Wt.

The design of the building now proceeds by applying the seismic shear to the floors using the traditional load distribution from seismic analysis discussed in Chapter 13 and then making sure that there is a competent load path to carry those loads to the ground. We are required to increase the forces by 33% for the steel connections, diaphragm design and connections in anchor rods. There are also requirements for checking the drift limitations and using accidental eccentricity.

When doing seismic design using the low seismic risk procedures of NBCC 2015 it is not necessary to refer to the other provisions of S16 Clause 27, including the requirements for capacity design, as long as load path for the lateral load is provided. If the designer prefers, the provisions of the low seismic requirements can be replaced by computation of seismic forces using the high seismic provisions of NBCC 2015 and the resistance systems of Clause 27.

CHAPTER 20
DESIGN OF DIAPHRAGMS FOR STEEL BUILDINGS

It is fundamental that the seismic forces be transferred from parts of the building to the seismic resisting elements. In order for this to occur and to stabilize the top flanges of beams, the building must have a diaphragm. Diaphragms are analogous to beams; the steel deck acts as the web of the beam to transfer shear forces while the diaphragm chords must carry the bending moments. This is shown in Figure 20.1.

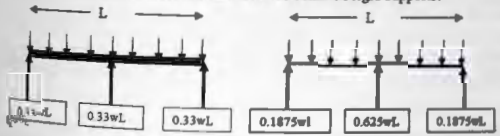


Figure 20.1. a) Roof diaphragm components. b) Equivalent beam components

Diaphragms must be evaluated separately in both of the two principal directions. (Sometimes this is done by inspection by evaluating the deck for the long span direction and using the same chord size along all sides.)

20.1 DIAPHRAGM FLEXIBILITY

Most of the guides to diaphragm design give a description of the flexibility of the diaphragm and how this has an effect on the distribution of forces in the structure. If a diaphragm has support at each end, then the flexibility of the diaphragm is of no consequence as the distribution of forces is defined by statics. If, however, the diaphragm is supported in three or more places, then it becomes statically indeterminate and the flexibility of the diaphragm can have an influence on the distribution of force to the supporting brace bays. Our analogy of a beam is again useful in this discussion. If we have a very rigid beam supported on springs, then the beam will move as a rigid body and the forces on the springs will be more evenly distributed. If, however, the beam is flexible, then the load on the springs will be determined more on the loads of the tributary area. In the latter case, the deformation of the brace bays becomes less important to the reactions and the system can be modelled as a flexible beam on rigid supports.



Rigid beam with three equal spring supports

Flexible beam with three equal rigid supports

Concrete floors and concrete on metal deck are generally regarded as rigid diaphragms, while diaphragms that consist of metal deck only are often regarded as being flexible diaphragms without justifying calculations. In reality, some metal deck diaphragms may calculate out to be rigid. The distribution of forces to the brace bays is less critical in brace bays designed with capacity design principles, as overloads on one brace bay will lead to increased deflection without failure, and this increased deflection will lead to a redistribution of force to lighter loaded brace bays. The use of state-of-the-art computer programs such as ETABS will provide methods for taking account of the flexibility of the diaphragm, but this is seldom done in small buildings. Flexible diaphragms often have forces distributed to base bays on the basis of tributary areas with modifications based on judgement. This is one aspect of structural engineering with many grey areas.

EXAMPLE—DETERMINE SEISMIC LOAD OF A ROOF DIAPHRAGM

Ignoring at this time the effect of eccentricity, determine the seismic load perpendicular to the long side of the roof diaphragm for the two-storey 40m x 22.5m brace bay building, as shown in Figure 20.2.

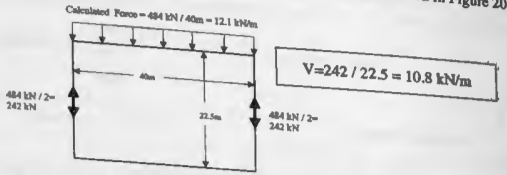


Figure 20.2. Seismic load perpendicular to long side of roof diagram

Solution:

The load on the roof has been found to be 484kN.

$$F_{Calculated} = \frac{484kN}{(40m)} = 12.1\frac{kN}{m}$$

Taking the load and divide by the diaphragm dimension get:

Result:

The calculated factored force perpendicular to the long side of the diaphragm is 12.1kN/m. The resulting diaphragm shear is 242kN / 22.5m = 10.8kN/m

20.2 WIND LOADING ON THE DIAPHRAGM

Lateral loads and hence the resulting diaphragms even for low buildings located in low seismic risk areas may be governed by wind loading. Diaphragms should be checked for wind loading. Usually wind loading is determined using static procedures and the figures from the NBCC Supplement. Regardless of the source of the governing lateral load, there is still a requirement to detail the diaphragm and provide a system to transmit the lateral forces to the lateral force resisting system. For areas with very low seismic loads where capacity design will not be used, notional loads should also be considered in the diaphragm design.

EXAMPLE—DETERMINE FACTORED WIND LOAD

Determine the factored wind loading acting on the roof diaphragm of the sample building. Assume exposed terrain and a 0.5m high parapet. (For forces use NBCC commentary Figure I-7 with a roof slope of 0 degrees.)

From Figure I-7 for surfaces 1 & 4: $C_pC_g = 0.75 - (-0.55) = 1.3$

$V = (C_e = 1.0) * (q_{50} = 0.48kN/m^2) * (C_pC_g = 1.3) * (0.5m + 4.5m/2) * (\text{Load Factor} = 1.4)$

$V = 2.4kN/m$ (Factored wind approx. 20% of calculated factored seismic shear force)

The factored wind loading is calculated as it provides a direct comparison to the factored seismic loading. In this case the wind force does not govern and the diaphragm will be reviewed and designed for seismic forces only. Factored wind loading is as shown in Figure 20.3.

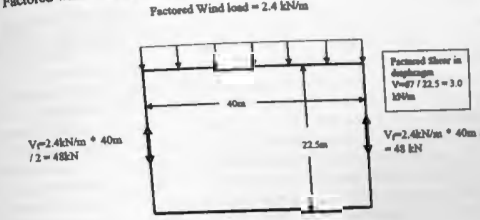


Figure 20.3. Factored wind loading

20.3 CODE REQUIREMENTS FOR FORCES ON THE DIAPHRAGM

The NBCC 2010 building Code has three separate clauses on diaphragms under Clause 4.1.8.15 Design Provisions. Clause 4.1.8.15.1 has the requirement that the diaphragm be designed for the governing of the following two loads:

1) forces due to loads … applied to the diaphragm are increased to reflect the lateral load capacity of the SFRS, and accounting for discontinuities and changes in the stiffness in these elements, or
2) a minimum force corresponding to the design based shear divided by N for the diaphragm at level x. This latter requirement will govern diaphragm at lower levels of multi-storey buildings.

Under NBCC 2010 Clause 4.1.8.15.2 was added to give special requirements for diaphragms that have been shown to exhibit some ductility. There are two types of diaphragms that have the potential of meeting this requirement; wood diaphragms and appropriately detailed steel diaphragms. To paraphrase Clauses 4.1.8.15.2 and 3 we are permitted to allow the roof diaphragm to yield and then design the diaphragm for forces corresponding to $R_dR_o = 2.0$ without considering the overstrength of the seismic force resisting system provided:

- The building is less than 4 storeys high.
- The diaphragm is detailed according to the applicable referenced design standards to exhibit ductile behaviour.
- Any struts, collectors, chords and connections to the seismic force resisting system must be designed for the lesser of the capacity design forces or the $R_dR_o = 1.3$ forces.

In S16 the requirement that the diaphragms are designed to resist the capacity design forces of the seismic force resisting system is covered in Clause 27.1.2(e) which requires that diaphragms and collector elements are capable of transmitting the capacity design forces at each level to the seismic force resisting system.

S16 does not permit the use of ductile diaphragms to reduce forces considered in diaphragm design unless the building is designed under "Conventional Construction" of S16 Clause 27.11 with specific requirements that allow the use of the $R_dR_o = 1.5*1.3 = 1.95$ cut-off force for diaphragms. Clause 27.11 requires that to use for the $R_dR_o = 1.95$ forces the diaphragm must be detailed such that the connection mode is ductile. The commentary contained in the *CISC Handbook* provides additional guidance:

> The failure of steel deck diaphragms is typically controlled by failure of the connections between the individual deck sheets and between the deck sheets and the supporting structure. Diaphragms designed and constructed using connections that have been shown by testing to be ductile can be designed using the factored forces calculated for conventional construction while those diaphragms with connections that have not been shown to be ductile should be designed using forces calculated using $R_dR_o = 1.3$. Button-punched side-lap connections or arc-spot welded connections commonly used for steel decks have not shown adequate ductile behaviour under cyclic loading. Research investigation into diaphragm designs for more ductile response is underway. Test results reported by Essa *et al.* (2003), Tremblay *et al.* (2004), and Hilti™ (2007) suggest that diaphragms made of thin steel deck sheets (0.76mm and 0.911mm) with power-actuated frame fasteners and screwed sidelaps can accommodate some inelastic deformations through screw tilting and bearing and tearing of the steel deck sheets at frame fasteners. Welded connections with washers when properly fabricated, can also sustain elastic deformation demand (Peuler *et al.*, 2002) although this approach is generally less appealing from a practical standpoint.

In flowchart form for an $R_d = 1.5$ system, this is as shown in Figure 20.4.

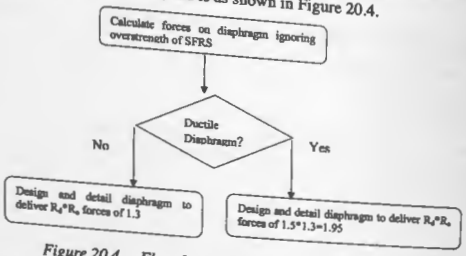


Figure 20.4. Flowchart for design of an $R_d = 1.5$ system

20.4 DEVELOPMENT OF DECK DIAPHRAGM PHILOSOPHY 2001 TO PRESENT

Over the past decade the philosophy of designing deck diaphragms has developed as research has advanced on this topic and engineers have attempted to put the research into practice. Two types of diaphragms have emerged from this work: (1) the traditional "brittle" diaphragm and (2) the "ductile" diaphragm.

1) ***Brittle Diaphragms*** – Recent research on the post-elastic behaviour of deck panels has shown that diaphragms connected to the steel frame using puddle welds and button punched at the side-laps are very brittle and respond poorly to overloading. Videos of the tests show the deck material tearing away from the welds and the deck sheets separating at the seams as the button punching lets go. This is disturbing for most designers of steel buildings who have many examples of this type of diaphragms in their inventory. The research shows that diaphragms with *either* puddle welds or button punching will behave in a brittle manner. It should be noted that there is no field evidence of these problems but we should keep in mind that reviewing the lab testing on moment connections after the Northridge earthquake indicated that there were problems with moment connections long before it showed up in the field.
2) ***Ductile Diaphragms*** – The research that has investigated diaphragms above the yield limit has shown that the diaphragm will behave in a much more ductile manner if they are connected to the steel frame using powder activated pins or self-tapping screws and if they are connected at the deck seams by screws.

20.5 A BRIEF HISTORY OF SEISMIC DESIGN OF DIAPHRAGMS IN CANADA

Up to 2005 roof decks used as diaphragms were welded to the supporting steel with puddle welds and the sheets were connected to each other by "button punching" which involved a crimping of the lap splices at the sides of the sheets. The system was reasonably easy to fabricate and tables of deck diaphragm capacity using unfactored shears were easily available from the deck manufactures. In 2005 tests performed at École Polytechnique showed that under dynamic loading that overloaded the deck capacity the deck sheets would come apart and the welds between the deck and the supporting steel would fracture. This resulted in a failure of the diaphragm – the seismic loads from the inertia of the roof weight as the building moved could no longer get to the brace bays and there was no longer lateral support for the compression flange of the roof beams. Also in 2005 a new edition of the National Building Code introduced requirements for higher shears and emphasized that the diaphragm should remain elastic. This was a perfect storm for the deck industry – not only were the loads increasing but there was concern that the traditional methods to provide the resistance were no longer effective. A period of rapid research followed in both university facilities in Canada as well as the private facilities of Hilti™ in both the United States and Liechtenstein. The result of this research was the development of side-lap screws and pins to connect the deck to the supporting material. While there were some initial issues with this fastening the pins and the side-lap screws improved to the point that by 2010 the use of mechanical fastening for roof deck was considered to be essential in high seismic risk areas and new construction instead of using button punched decks was now using side-lap screws and welding was restricted to only occasional use on 1.22mm and 1.52mm thick roof deck. One improvement that came from continued testing and field studies was the use of screws instead of pins at the deck locations where the end laps had extra layers of deck that had to be connected to the supporting steel. Another improvement was the use of better capacity side-lap screws which helped improve the shear capacity of the deck. Designers were now requiring thicker chords on the joists that the deck was being fastened to and deck pins had greater shank diameters so that thicker roof decks were not able to damage the pins.

20.6 INCREASING THE DIAPHRAGM FORCES TO ACCOUNT FOR CAPACITY DESIGN

The current S16 code seismic requirements have changed considerably since the 1994 edition of the steel standard. S16 now firmly adopts the philosophy of capacity design requiring that systems designed with R_d-values of 2 or greater be detailed such that this yielding can occur. S16 Clause 27.1.2 requires that for all systems with R_d of 2 or greater have their diaphragms designed to carry the over strength forces of the seismic force resisting system.

This principle applies to systems with an R_d of 2 or greater as it is desired that, with these systems, the diaphragms deliver to the seismic force resisting system the forces that are needed to yield the braces or other yielding elements of the seismic force resisting system. The diaphragms of R_d of 2 or greater systems will ideally be designed for the capacity of the seismic force resisting system but may be limited to the forces corresponding to $R_d = 1.3$. Showing both the R_d of 1.5 system and the R_d of 2 or greater system, we get the flowchart in Figure 20.5.

Note that for the R_d of 2 or greater systems there is no reduction in the design forces that can be used if the diaphragm is detailed as a "ductile" diaphragm. This reflects the desire of the Code that, for

buildings with ductile seismic systems, the yielding element should be the seismic system itself. If both the diaphragm and the seismic system are trying to yield at the same time, then the force distribution in the building and its resulting response may be very different from that considered by the designer. Where possible the designer is encouraged to build ducility into the diaphragm and use good detailing practice but the design forces on the diaphragm are unchanged.

For systems having an R_d of 2.0 or greater where there is a large overstrength in the seismic system, there is ongoing debate on the use of a limiting value of the overstrength. This might occur in a masonry building designed and detailed to $R_d = 2.0$, which might occur where a designer wishes to design a fire hall and comply with the minimum R_d of 2.0 for these structures but still wants to use masonry. Some designers are advocating that the diaphragm forces should be limited to those corresponding to $R_d = 1.5$ while others are proposing that $R_d = 1$ forces be used.

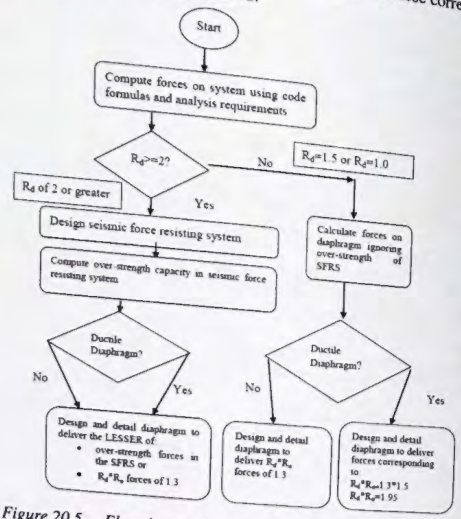


Figure 20.5. Flowchart of decisions for R_d of 2 or greater systems

EXAMPLE—DETERMINE DIAPHRAGM SHEAR AT PUSHOVER CAPACITY OF A BRACE BAY

For a 22.5m x 40m two-storey brace building, determine the factored force perpendicular to the long side of the roof diaphragm including effects of overstrength of the SFRS. Compare this to the limiting value of load for $R_dR_o = 1.3$. The braces have a pushover capacity of 481kN.

Solution:

The design forces in the brace bays at the roof level including the effect of eccentricity have been found to be 290kN. The pushover capacity of the brace bay is found using capacity design principles to be 1.66 times the applied forces. The force at the roof at each brace bay is therefore 1.66*290kN = 481kN. The effective force on the diaphragm at brace overstrength is therefore (481kN *2) / 40m = 24.1kN/m

The free body diagram of the roof at the **capacity of the SFRS** is as shown in Figure 20.6.

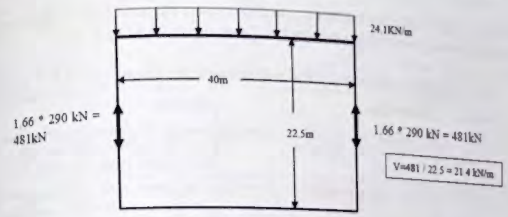


Figure 20.6. Free body diagram showing capacity of the SFRS

The limiting design load on the diaphragm is computed at $R_dR_o = 1.3$ and is found as using a linear ratio of the calculated forces to the appropriate R_dR_o values as follows:

$$\text{Load perpendicular to long side} = W_f = 12.1\text{kN/m} * \frac{R_dR_o=3.9}{R_dR_o=1.3} = 36.3\text{kN/m}$$

In our case it is obviously preferable to design for the forces calculated on the diaphragm increased for the overstrength of the system. In other systems where there is not a well-defined yielding mechanism it may be more advantageous to design for the limiting forces.

20.7 DIAPHRAGM CONNECTORS

There are two types of diaphragm connections to consider and these connections will have an influence on the behaviour and resistance of the diaphragm to shear loads. The first connection to consider is the connection of the diaphragm to the beams, joists, chord angles and drag struts while the other is the connection between diaphragm sheets.

Sheet to support connection: There are at least four options for connecting the sheets to beams, joists, chord angles and drag struts. These connections are as follows:

a) Puddle welds typically 3/4-inch (19mm) in diameter. Recent tests have shown that this system will behave in a brittle manner if the deck is overstressed. Tests have shown the deck tearing at the weld during the load reversals. While some design firms have stopped using puddle welds to connect roof deck to structure, the use of puddle welds is still the preferred connection method for floor decks. Testing is ongoing to determine if weld problems exist on roof decks using 1.22 and 1.52mm material.

b) Puddle welds where the puddle weld capacity is enhanced by the addition of a washer prior to welding. The weld is therefore between the inner face of the washer, the sandwiched deck and the supporting beam. This fastening system has worked well in tests but has proved to be unpopular with workers in field installations and is now not used.

c) Shot in pins using a powder actuated fastening system. Pins used for connecting deck to bar joists are the Hilti™ X-EDN19 or X-EDNK22 pins. The frame members that the deck is being fastened to can be 3mm to 9.5mm thick. Pins used to connect deck to structural steel including edge angles having a thickness of greater than 6mm are the ENP2-21 L15 or ENPH2-21 L15 fastener. Deck pins are installed with a stand-up tool. The procedure is popular with contractors as it the installation can take place in wet weather. Field tests have shown the installation to be almost twice as fast as the traditional puddle welding.

d) Screwed fasteners include two types of connections that are proposed, depending on the thickness of material that the deck is being connected to. These are shown in Table 20.1.

Table 20.1. Screw types for different base material thicknesses

Base Material Thickness	Screw Type	Remarks
3mm to 5.3mm	#12-14 with #3 drill point	Suitable for cold rolled joist chords and thinner gauge hot rolled chords.
6mm or more	#12-24 with #5 drill point	Suitable for connections to beam flanges but may require pre-drilling.

Sheet to sheet connection: There are at least three options for connecting the sheets to each other

1) Button punching in which the side-laps of the deck are synched together using a hand-operated tool. Recent tests have shown that this system does not work well if the deck overstressed. This writer is unaware of any design firms in Vancouver still using button punching to connect roof deck sheets together.
2) Side-lap welding is not very common in our area and has field difficulties especially for thin sheets.
3) Side-lap screws preferably placed using a stand up fastener. Lab tests show that this method of connecting the sheets is superior to button punching and field experience has been successful. The presently preferred side-lap screw is the Hilti™ SLC (side lap connector). Field tests have shown that the screws are economical and the stand-up tool is not only easier on workers than button punching but the screws can be installed almost as fast as button punching.

20.8 SHEAR CAPACITY OF THE DIAPHRAGM

The shear capacity of the deck is determined from tables or computer programs that take into account the following factors:

a) The type of connection at the sides of the deck (button punching, side-lap screws or seam welds)
b) The spacing of the connection at the sides of the sheets
c) The type of the connectors between the deck flutes and the supporting members (puddle welds, pins or screws)
d) The spacing of connectors between flutes and the supporting members
e) The centre-to-centre spacing of the supporting members
f) The gauge (thickness) of the deck
g) The deck profile (3-inch or 1.5-inch)
h) Buckling of the deck, which is principally a problem for lighter thinner decks at longer spans
i) The resistance factor (Ø) used. These should be appropriate for the current version of S136 and the fastening system used

20.9 DECK TO STRUCTURE CONNECTION PATTERNS

The designer will specify the number of connectors that are placed between the deck and the supporting structure. This can range from a connector at each side of the sheet and every second flute to two at each side and one connector in each sheet. The patterns are usually described as being the width of the sheet and the number of connectors per support. For example, a 36/9 pattern means the sheet is 36-inch wide and has 9 pins (two at each side of the sheet and one at each flute). Figure 20.7 show typical connector patterns.

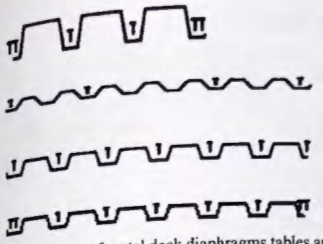


Figure 20.7. Typical deck connector patterns
Source: Adapted from the Hilti™ Diaphragm Program

The shear capacity of metal deck diaphragms tables are published by the deck manufactures for their deck and the CSSBI or can be calculated from computer programs such as the Hilti™ PROFIS program. The capacities typically are based on the Tri-Services method which takes into account various failure methods of the deck. The values computed by the Tri-Services method are then reduced by the S136 resistance factors shown in Table 20.2.

Table 20.2. Safety factors and resistance factors for diaphragms

TABLE D5
Safety Factors and Resistance Factors for Diaphragms

Load Type or Combinations Including	Connection Type	Limit State: Connection Related Ω_d (ASD)	Connection Related ϕ_d (LRFD)	Connection Related ϕ_d (LSD)	Panel Buckling* Ω_d (ASD)	Panel Buckling* ϕ_d (LRFD)	Panel Buckling* ϕ_d (LSD)
Earthquake	Welds	3.00	0.55	0.50			
	Screws	2.50	0.65	0.60			
Wind	Welds	2.35	0.70	0.65	2.00	0.80	0.75
	Screws						
All Others	Welds	2.65	0.60	0.55			
	Screws	2.50	0.65	0.60			

Canada

For Canadians using Limit States Design (LSD) and seismic loading, this reduces to the resistance factors as shown in Table 20.3.

Table 20.3. Resistance factors

	Connection Related	Panel Buckling
Decks connected by welds or button punching or both.	0.5	0.75
Decks connected to the frame by screws or pins **and** at side-laps by screws.	0.6	0.75

The resistance factors to be used are often an input in the computer programs used to calculate diaphragm capacity and should be checked to see that they are matching the factors from Table 20.3. These resistance factors have changed three times in the past seven years (see Appendix 3) and it is important to make sure that the tables or program that you are using has the correct resistance factors built in.

The Hilti™ PROFIS diaphragm programs are the current state of the art for determining diaphragm shear capacities with updates to the program as standards change and connector capacity increases. The program can handle screwed side-laps and pinned and screwed and welded deck to structure connections. While the tables and computer programs usually give the deck capacity in tabular form it is much more enlightening to plot this information to determine the relative importance of the effects. By taking the data from the Hilti™ program and it is possible to produce charts that allow easy comparison of various effects such as changing the thickness of the deck, the fastener spacing and the type of fasteners used.

A sample deck shear factored resistance chart is shown in Figure 20.8.

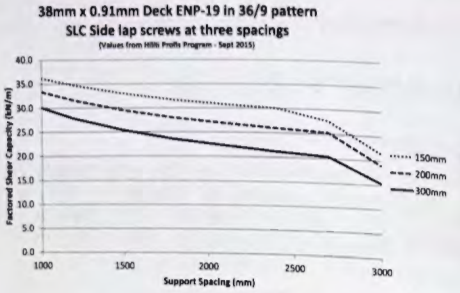


Figure 20.8. Deck shear capacity 38mm deck at various fastener spacings.

The deck capacity chart in Figure 20.8 shows that the spacing of the deck supports has a significant effect on the deck shear capacity as does the spacing of the side-lap screwing. While the joist or beam spacing is usually controlled by vertical load factors it is worth noting that greater joist spacing will reduce the diaphragm capacity. At high values of shear and long spans the deck buckling restricts the shear capacity of the diaphragm this is particularly true for 38mm deck and is demonstrated in the capacity chart shown Figure 20.8 with the sharp dip down in shear capacity when the 38x0.91 deck with side-lap fasteners at 150mm o/c reaches a span of 2.5m. The deck buckling has less effect on the deck fastened at 300mm o/c as the shear capacity is limited by the shear transfer mechanisms at the side of the sheets and is not carrying the shears that lead to buckling. For a roof deck supported at 2000mm on centre we get a capacity of 22.6kN/m with a sidelap connector spacing of 300mm and 31.0 kN/m with a sidelap connector spacing of 150mm.

Examining the effect of 38mm deck thickness in Figure 20.9, we can see that, the thicker the deck, the more effective it is at resisting shear but the effect is less pronounced after the deck reaches 1.22mm with the shear resistance of 1.22mm and 1.52mm decks being very similar. Plotting the shear capacity for 76mm at various thicknesses as is done in Figure 20.10 shows that the shear capacity for 76mm deck is lower than for 38mm deck. This effect is especially pronounced if we look at what the expected support spacing would be for a 38mm x 0.91 deck supported at 1800mm o/c we have a factored shear resistance of 23.5kN/m while for 76mm x 0.91 deck supported at 2500mm o/c we have a factored shear resistance of 12.6kN/m.

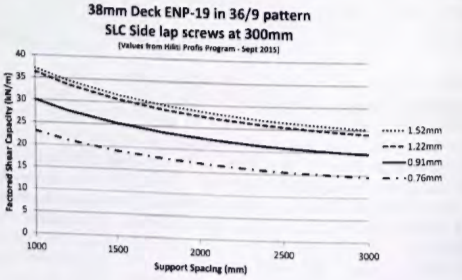


Figure 20.9. Deck shear capacity 38mm deck at various thicknesses.

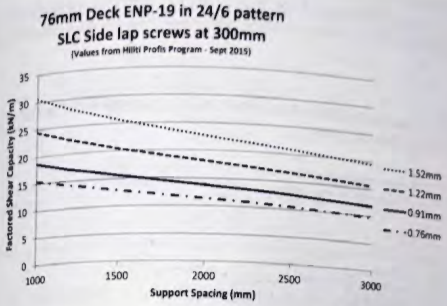


Figure 20.10. Deck shear capacity 76mm deck at various thicknesses

Since 2004 an extensive research program has been trying to address the issue of shear diaphragms under cyclic loading and while some of the early findings were disappointing recent tests and field experience with the latest deck fasteners have demonstrated that deck capacity has improved and the decks are performing in a more ductile manner and connector failures are not occurring from thermal loading during construction.

The use of side-lap screws and pins is beneficial not only from the standpoint of the ductility of the deck system but also because of the increased factored shear resistance of the deck when using the new fasteners.

This is illustrated in Figure 20.11, where four types of fastening have been used on a 38x0.91 deck. The greatest capacity is achieved with Hilti™ Pins and side-lap screws, while other fastening systems show reduced capacity depending on the type of fastener and the spacing of the deck supports. At least part of this is due to the different resistance factors that are used for the button punching and welding as opposed to the pins and side-lap screws.

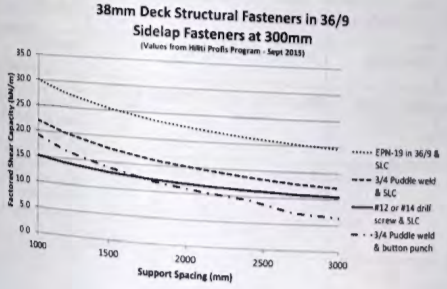


Figure 20.11. Comparison of deck mechanical and traditional fasteners

EXAMPLE—DETERMINE NECESSARY DETAILS FOR ROOF DIAPHRAGM

Continuing the example presented earlier, determine if the 38 x0.91 deck is sufficient and determine the button punching, puddle welding, and other details necessary. Joists supporting the deck are supported at 1.8m o/c.

Solution:

The diaphragm shear found earlier was determined to be a factored force of 481kN with the diaphragm width of 22.5m this gives 481kN / 22.5m = 21.3kN/m.

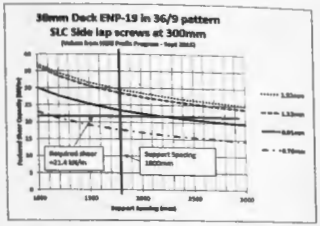


Figure 20.12. Selection of Deck for applied shear. Deck fastened with SLC at 300 o/c and ENP-19 in 36-9 pattern

Examining Figure 20.12 it can be seen that with a support spacing of 1.8m the required shear capacity of 21.3kN/m is achieved with a 38 x 0.91 mm deck with Hilti™ Pins in a 36-9 pattern and side-lap screws at 300mm on centre. Any increase in the shear would require going to 38 x 1.22 deck or by reducing the spacing of the screws at the side-laps. Diaphragm options are listed in Table 20.4.

Table 20.4. Diaphragm options

CONDITION	R_dR_o	DIAPHRAGM SHEAR	CONTROLLED BY	DIAPHRAGM
Brace Bay ignoring eccentricity and over-strength	3.0x1.3=3.9	10.8 kN/m	N/A	N/A
Brace Bay including eccentricity but not over-strength	3.0x1.3=3.9	12.9 kN/m	N/A	N/A
Brace Bay including eccentricity and over-strength Ductile Diaphragm	3.0x1.3=3.9	21.4 kN/m	Capacity design for brace bay	38x0.91 deck Pins in 36/9 pattern #12 screws at 150 o/c
Brace Bay including eccentricity and over-strength Traditional Diaphragm	3.0x1.3=3.9	21.4 kN/m	Capacity design for brace bay	38 x 1.22 deck puddle weld in 36/9 pattern. Button punch at 100 o/c.

CONDITION	R_dR_o	DIAPHRAGM SHEAR	CONTROLLED BY	DIAPHRAGM
Wind Load	Independent of system.	2.14 kN/m	Calculated factored wind load	If wind only (and most diaphragms work for this example.
Masonry or Tilt-Up Ductile diaphragm	Diaphragm designed for 1.5x1.3=1.95	21.6 kN/m	Calculated forces on R_d=1.5 system	38x0.91 deck Pins in 36/9 pattern #12 screws at 100 o/c
Masonry or Tilt-Up Traditional diaphragm	Diaphragm designed for 1.0x1.3=1.3	32.4 kN/m	R_dR_o=1.3	Can't get traditional diaphragm to work for these shears

20.10 MOMENT CAPACITY OF THE DIAPHRAGM

The moment capacity of the diaphragm depends only on the capacity of the chords. The metal in deck itself is not considered to take part in resisting the diaphragm moments. In one direction the deck is a series of bends that are trying to straighten out and in the other direction the deck is a series of segments having only a series of puddle welds at panel edges to connect tension. The moment in the diaphragm divided by the distance between the chords results in the design force in the chords. The chord size can be selected and detailed. In particular connections in the diaphragm, chord should be detailed to transmit the forces expected between joints in the chords. Angles are usually used as diaphragm chords but if there are edge beams they can be the diaphragm chords providing the following two conditions are satisfied.

1) The deck diaphragm must sit directly on the beam not be separated by a series of joist seats and
2) The beams must be detailed for the force expected when they act as diaphragm chords including appropriate moment / axial interaction and calling up of the forces to be used in connection design.

EXAMPLE—DETERMINE THE MAXIMUM BENDING MOMENT AND SHEAR ON THE DIAPHRAGM

Taking the previous example and using the seismic force of 24.1kN/m acting on the roof diaphragm perpendicular to the long side of the building, determine the maximum bending moment and shear on the diaphragm.

Solution:

The seismic force has been determined to be 24.1kN/m, giving the free body diagram shown in Figure 20.13.



Figure 20.13. Free body diagram

Maximum Shear = (40m /2) * 24.1kN /m = 481kN (in our case with only two braces in each direction and the braces being at the outside of the structure this is equal to the pushover force on the brace at the roof level).

Maximum Moment = $wL^2/8$ = (24.1kN/m)*(40m)^2 / 8 = 4820kNm

EXAMPLE—DESIGN DIAPHRAGM CHORDS

Continuing the example presented earlier, design the diaphragm chords for the roof.

Solution:

The diaphragm moment found earlier is 4820kNm. Diaphragm chord forces are found by dividing the diaphragm moment by the distance between chords.

$$C_f = T_f = \frac{4820kNm}{22.5m} = 219kN$$

Assuming an angle with F_y = 300 MPa and assuming that the deck and the adjacent wall fully supports the angle against buckling we can determine the area required:

$$\text{Area Required} = \frac{219kN}{(\phi = 0.9)*(F_y = 300000kN/m^2)} * 10^6 mm^2/m^2 = 811mm^2$$

This corresponds to an L76x76x6.4 with Area = 920mm^2. If the edge angle was attached to a tilt-up panel it might be assumed that there was sufficient restraint to prevent buckling and the area needed for tension is sufficient for compression. If the edge angle is not restrained consider compression capacity and assume deck restrains angle in one direction.

L76x76x6.4 r = 23.6mm kL/r = 0.9 * 1800 / 23.6mm = 68.6

For 300 MPa C_r / A = 188 MPa

A_{req} = 219kN / 188,000kN/m^2 = 1165 mm^2 **Use L 76x76x7.9** (A = 1150 mm^2)

For tilt-up buildings the detail for attaching the chord angle to the embeds of tilt-up panels calls for a 6mm weld so we should at least have an angle of thickness that exceeds this. Note that for deflection control or detailing reasons chord angles are often larger than is necessary purely from a strength standpoint. This makes it harder for the diaphragm to become the yielding element in the system.

20.11 CAPACITY OF DECK-TO-FRAME CONNECTORS, INCLUDING PUDDLE WELDS, SCREWS, AND PINS

If we are connecting the deck to other elements to transfer shear force, then we must have a value for the capacity of the puddle welds and #14 screws. The values given by VicWest® for calculation of deck details are widely used in the design community as there is little other guidance. The values given are in pounds and for allowable (unfactored) design. These are converted to kN and multiplied by a load factor of 1.5 in Table 20.5.

Table 20.5. Capacity of various deck-to-frame connectors

Deck	Puddle weld allowable shear load (lbs)	Puddle weld factored shear resistance (kN)	#12 or #14 Screw allowable shear load (lbs)	#12 or #14 screw factored shear resistance (kN)	X-EDN-19 or X-EDNK-22 Mechanical Fastener Allowable Shear Load (lbs)	X-EDN-19 or X-EDNK-22 Mechanical Fastener Factored Shear Resistance (kN)
0.76mm	863	5.8	332	2.2	500	3.33
0.91mm	1025	6.8	437	2.9	600	4.00
1.22mm	1380	9.2	780	5.2	785	5.23
1.52mm	1654	11.0	1076	7.2	975	6.51

Source: The values for puddle welds and screws are as set out in the February 1995 revision to the 1991 VicWest® *Steel Deck – Shear Diaphragm Design Manual*. The values for the Hilti™ connections are taken from the Hilti™ Steel Deck Fastening System 2009 North American supplement to their 2008 technical guide.

EXAMPLE—DETERMINE CONNECTION SPACING

Continuing the example of the 22.5m x 40m two-storey brace bay building presented earlier determine the required spacing of the connection of the 0.91mm deck at the perimeter angles. The deck is connected using Hilti™ X-EDN-19 fasteners.

Solution:

The diaphragm shear force adjacent to the 22.5m side chord angle was found earlier to be 481kN /22.5m = 21.4kN/m. Using the 0.91mm deck we find that pin connections are required at $\frac{4.00kN\ /\ fastener}{21.4kN\ /\ m} = 0.186m$. Say, 150mm.

The pins will be placed at 150mm at diaphragm edges. This spacing is quite tight and requires about twice as many pin fasteners as welds. Even if the calculated spacing is greater than 300mm, most designers would use a minimum edge fastener spacing of 300mm unless calculations showed a tighter spacing was required. The CSSBI recommends that even in low shear situations the perimeter fasteners should be spaced at no more than 900mm on centre.

20.12 VARYING THE DECK TO STRUCTURE CONNECTIONS AND SIDE-LAP CONNECTIONS

As we have determined the spacing of deck to structure connection and side-lap connection from the worst conditions it would seem that savings could be made by decreasing these connections in areas of lower stress. This procedure is not recommended by the CSSBI for the following reasons:

1) Due to building eccentricities and other distribution issues it is possible that the lighter loaded areas may get more loading than expected. Research at the University of Alberta has shown that the shear force diagram is not linear as we move away from the support but is more of a parabola with the maximum value at the end of the diaphragm not decreasing as fast as a linear distribution would predict.
2) The varying of fastener and puddle welding patterns adds complexity to the construction and field inspection activities.

If the designer decides to overrule the foregoing and use different fastener and welding patterns on lighter loaded areas of the deck the following should be observed:

a) Do not have too many different fastener spacing areas, a maximum of three different areas would keep the complexity within reason.
b) Draw a key plan that clearly shows where the different fastener and puddle welding areas exist
c) Be conservative in the cut-off of closer spaced fasteners and fasteners such that partial loading and eccentricities can be accounted for.

20.13 DECK DIAPHRAGM DEFLECTIONS

The diaphragm deflection must comprise two parts the bending deflection from the axial shortening of the diaphragm chords and a shear deflection from the deck. For a simple span deck this is covered in the following formula:

$$\Delta = \frac{5WL^4}{384EI} + \frac{q_{avg}(L/2)}{G' \times 10^6}$$ Where L is the span of the diaphragm,

q_{avg} = average shear in the diaphragm between the support and midspan.

The deck deflections for the roof deck of the two-storey 40m x 22.5m building are covered in Appendix 2.

As with all seismic deflections the deflections of deck are computed with the computed seismic loads and then multiplied by $R_d R_o / I_E$ to give a realistic estimate of the deflections. For our 40m x 22.5m diaphragm the computed diaphragm deflections are 49mm due to bending and 62mm from shear for a total of 111mm. For a normal importance building this is close to the limit of $0.025h_s = 0.025*4500mm = 113mm$.

Due to the shear capacity of the deck system and the deflection of the diaphragm it is desirable to have diaphragms that have low aspect ratios. A diaphragm that is almost square (aspect ratio of 1.0 to 1.5) will be much easier to design than one that is getting very rectangular with an aspect ratio of 2.5 to 3.0. The CSSBI recommendations depend on the type of walls and the rigidity of the diaphragm. CSSBI recommends a maximum aspect ratio of 2 for flexible diaphragms with masonry walls to a maximum of 5 for rigid diaphragms with flexible walls.

Sometimes situations arise where the length of a diaphragm is much greater than its width. This leads to quite slender diaphragms with a high aspect ratio (length / width). If calculations for diaphragm shear and deflection are executed it will often be found that diaphragms with an aspect ratio of 3 or greater are difficult to justify. In such cases it would be worthwhile considering adding angle or flat-bar lacing to convert the diaphragm into a horizontal truss to carry shear loads.

20.14 FINALLY, CALL UP THE DIAPHRAGM REQUIREMENTS ON DRAWINGS

A very important final step is making sure that the diaphragm detailing required is called up on drawings as shown in Figure 20.14. This would include:

- The deck thickness – the diaphragm design process might have required an increase in the deck thickness.
- The type of connector and fastener pattern between the deck and the supporting members.
- The type of side-lap connector and the spacing of side-lap connectors.
- The perimeter chord angle and that it be continuous – a splice detail is also recommended.
- The spacing of deck connectors between drag struts and perimeter angles parallel to the direction of the deck flutes.

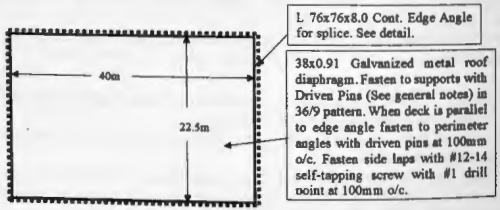


Figure 20.14. Galvanized metal roof diaphragm detailing

Various typical conditions exist with most diaphragms that it is desirable to have some typical details. Showing the typical details on the drawings at tender time means that contractor knows what is expected and the desired details will be obtained without extra charges or compromising the design at the erection stage.

20.15 DIAPHRAGMS FOR TILT-UP BUILDINGS

The use of steel deck diaphragms in tilt-up type buildings is common in Canada including seismic regions such as the lower mainland. The higher forces that have come with NBCC 2005 and the requirement that the diaphragm develop the capacity of the force resisting system has created significant challenges to designers of tilt-up structures. Tilt-up structures designed in accordance with A23.3 have been given an R_d of 1.5 and an R_o of 1.3. In previous codes the R-value assigned to tilt-up was often 2.0. Using an R of 2 or R_d of 1.5 it is difficult to find a yielding mechanism that produces SRFS capacity forces on the roof diaphragm less than the normal bail-out values specified in S16 of $R_d R_o = 1.3$.

Sample connections for a deck diaphragm chord to embed plates in the tilt-up panels are detailed in Figure 20.15. These conditions occur where the tilt-up panel marks the edge of the diaphragm. The 6mm weld requires that the chord angle is at least 6.4mm. The detail shows weld between the embed plate and the angle to be continuous across the full width of the embed plate. Sample diaphragm chord connections to tilt-up panels are shown in Figure 20.16.

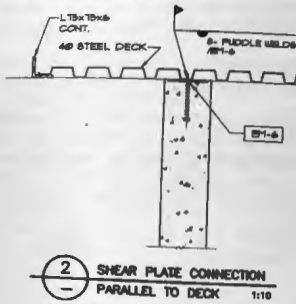


Figure 20.15. Connection of the panel when the diaphragm passes over a tilt-up panel may occur under some circumstances, the detail showing the connection of the diaphragm is given at left

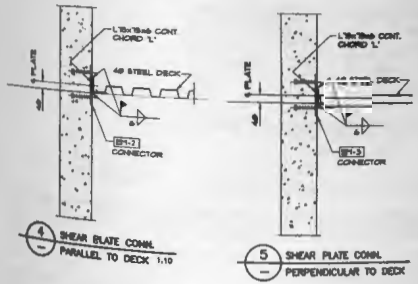


Figure 20.16. Sample diaphragm chord connections to tilt-up panels

20.16 ROOF DIAPHRAGM CHORD SPLICE

With the exception of very small roofs, the edge angles will be formed from shorter segments requiring the chords to be spliced. A typical note is to require the diaphragms to be spliced for full strength, but this can lead to varying results depending on the interpretation by the contractor of what constitutes full strength. A clearer option is to include a detail such the one shown in Figure 20.17. This detail gives directions on how to splice the chords and would have to be modified depending on the force in the chord.

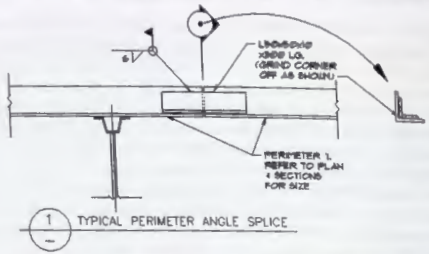


Figure 20.17. Typical detail requiring diaphragms to be spliced for full strength

20.17 CONNECTION OF THE DIAPHRAGM CHORD TO A HIGH FLUTE OF THE DECK

Another detail that is useful to have on the drawings is one that gives a solution to connecting the roof deck to the chords in the case where the deck flute is high at the location of the diaphragm chord. Just as it is important that the beam web be connected to the flange, the deck must be connected to the diaphragm chords. Part of the reason to show this detail is to make it desirable to the contractor to lay out the deck in a way that this detail does not have to occur. One solution to this problem is shown in Figure 20.18.

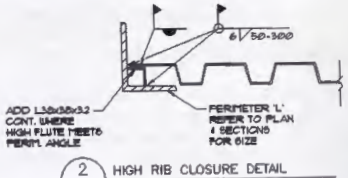


Figure 20.18. Diaphragm chord detail

20.18 SHEAR LUG DETAILS

Shear lugs are required where the deck has been separated from the beams at the top of brace bays by joist seats. Shear lugs bring the load from the deck to the level of the top of the brace bay and generally consist of bent plates welded to the beam below and to either the deck or chord angles. The shear lugs prevent the joist seats from trying to carry the shear force – something they are not very capable of doing. In tilt-up buildings these details would be necessary where a beam extends out from a wall to act as a drag strut or shear collector (see the figures on diaphragm with re-entrant corners discussed later in this chapter. The

details shown in Figure 20.19 are for deck fastening with puddle welding – if the deck is to be fastened with pins these details would be modified to show pins as the fastening should be consistent.

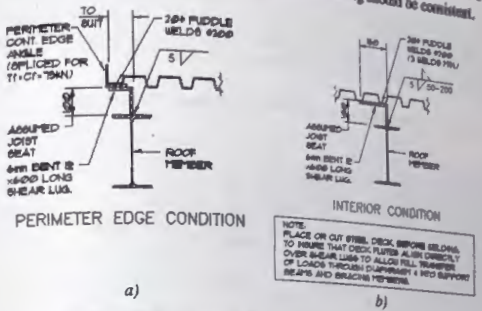


Figure 20.19. Typical details for shear lug connectors

20.19 PROBLEMS WITH REAL DIAPHRAGMS

Real diaphragms are often not purely rectangular and on one level. They frequently have changes in elevation, indentations that interrupt diaphragm chords and have holes. We will examine a few of these problems and some possible solutions.

20.20 INTERRUPTING DIAPHRAGM CHORDS

The diaphragm chords must provide a continuous path for the forces from the diaphragm moment such that the diaphragm can span from one brace bay to the next. Unfortunately the building form may require a bite out of the side of the diaphragm to accommodate an entrance feature or loading dock. Unfortunately when this occurs the diaphragm chords are often interrupted. The seriousness of this situation can be seen in Figure 20.20, which shows a beam and mechanical duct. Obviously the bottom flange of the beam cannot be cut to accommodate the mechanical duct and another satisfactory structural solution must be found. Note that a similar situation may get overlooked in diaphragm design.

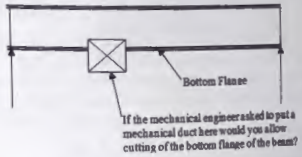


Figure 20.20. Beam cut by a mechanical duct in an unacceptable manner

The situation of the missing section of the diaphragm chord may be addressed in different ways. Some possible solutions include:

- Providing in plane trusses to transfer the chord forces around the opening as shown in the diagram. This situation can become tricky especially if the tendency of the in plane trusses to try to twist

are taken into account. The reduced distance between the diaphragm chords must also be taken into account when computing the force in these chords.

- Examine if the diaphragm chord can run through providing only an opening at the near the roof edge rather than at the roof edge. The diaphragm chord that does run through will be unsupported in the gap and must be designed to carry the compression force through this unsupported length.
- Review with the architect if the section in the roof must be removed.

Two solutions for a diaphragm with a interrupted chord are shown in Figure 20.21.

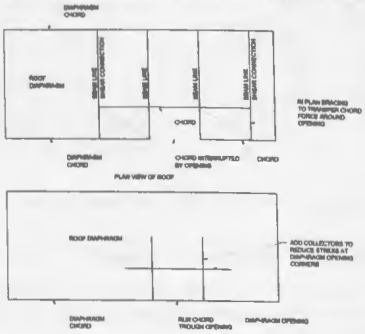


Figure 20.21. Two solutions for a diaphragm with a interrupted chord

20.21 STEPPED DIAPHRAGMS

Stepped diaphragms can often cause a problem as there must be a way for the forces from each diaphragm to be transmitted to shear resisting elements. In the diagram shown in Figure 20.22, the roof is stepped with a lower roof at the front of the building. There are brace bays going to the ground only at the front and rear walls.

A method must be provided to get the diaphragm shear forces from the upper diaphragm to the lower diaphragm and to make the chords continuous. Figure 20.22 shows these elements being added. From a structural engineering standpoint it would be more efficient to have a brace bay at the diaphragm step but this may conflict with the building function and not be possible.

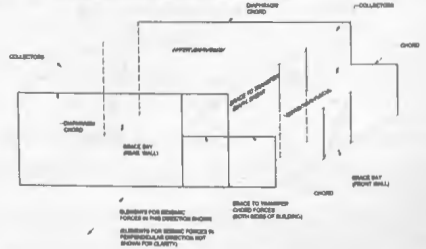


Figure 20.22. Stepped diaphragm showing the structure necessary to provide a complete load path

20.22 DIAPHRAGMS WITH RE-ENTRANT CORNERS

Diaphragms for roofs that are not rectangular will often have re-entrant corners such as the re-entrant corner shown in Figure 20.23. These corners prove to have undesirable tearing behaviour under earthquake loading unless care is made to detail the corner bars to carry the force concentrations into the deck. The drag strut should be of sufficient length that the deck connecting to it is not overstressed.

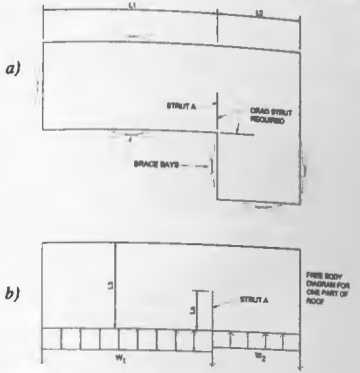


Figure 20.23. Typical re-entrant corners for roof diaphragms that are not rectangular

20.23 CANTILEVERED DIAPHRAGMS

Diaphragms that are cantilevered can be a problem from the standpoint of deflection at the tip and chord forces at the root of the cantilever. Design the cantilever deck using the principles demonstrated earlier. Care should be taken in developing the chord forces.

20.24 FLOOR DECK DIAPHRAGMS

The principles for floor deck diaphragms are similar to those for roof diaphragms, there must be chords to carry the moment but sometimes these chords are formed from reinforcing steel in the deck but the principles requiring continuity still apply. The shear of the diaphragm can be computed from concrete formulas for V_c and V_s using the reinforcement in the concrete on the deck. It is desirable to use Nelson Studs on collectors and drag struts to provide a more positive transfer of shear loads from the deck to seismic elements. Nelson Studs should be used to transfer shear from the deck concrete to the diaphragm chords if composite type deck has not been used.

20.25 DEFLECTION OF ROOF DECK DIAPHRAGMS

It is necessary to consider both the bending deflection of the diaphragm and the shear deflection of the diaphragm. The bending deflection of the diaphragm is a function of axial shortening of the chord members. The computation of the shear deflection requires consideration of the fastening and deck properties. Roof deck diaphragm tables include a factor F the flexibility factor that is used in determining the deflection of the deck. To return to our beam analogy the deflection due to bending can be found using the moment of inertia of the cords about the centre of the diaphragm but the flexibility factor determines the shear

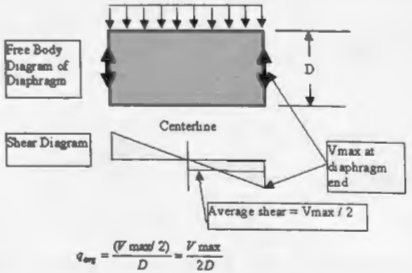
deflection. As with beams that have depth approximately equal to the span the shear deflection can be a significant portion of the total deflection.

For a simple span diaphragm uniformly loaded such as our example the deflection can be calculated as follows:

$$\Delta = \frac{5WL^4}{384EI} + \frac{q_{avg}(L/2)}{G' x 10^6}$$

Where L is the span of the diaphragm

q_{avg} = average shear in the diaphragm between the support and the centreline and is illustrated in Figure 20.24.



$$q_{avg} = \frac{(V\ max)/2)}{D} = \frac{V\ max}{2D}$$

Figure 20.24. Average shear in the diaphragm between the support and the centreline

G' is the shear flexibility of the deck and is computed as follows:

$$G' = \frac{K_2}{K_4 + 0.3(D_{xx} / Span) + 3(K_1)(Span)}$$

The coefficients can be found in the *Steel Deck Institute Manual* and those that are variables are output as part of the Hilti™ Program.

The values are as follows:

K_1 is obtained from the Hilti™ program and may be converted from imperial to metric units by multiplying by 3.387. For 38x0.91 deck side-lap fastened at 150mm o/c with joists spaced at 1800mm o/c, the value of $K_1 = 0.290 \times 3.387 = 0.982$

K_2 is dependent on the deck thickness as shown in Table 20.6.

Table 20.6. K_2 factor for deck deflection calculation

Deck Thickness	K_2
0.76mm	158 kN / mm
0.91mm	191 kN / mm
1.22mm	252 kN / mm
1.52mm	319 kN / mm

K_4 is a function of deck profile but for all roof decks = 3.78

D_{xx} is the warping constant, which is a function of the fastening pattern and the deck thickness, with units of metres these values are shown in Table 20.7.

Table 20.7. Flexibility factors for deck deflection computation

Deck	Fastening Pattern	0.76mm	0.91mm	1.22mm	1.52mm
38mm	36/9	39	30	19	14
38mm	36/7	39	30	19	14
38mm	36/5	231	173	113	80
38mm	36/4	327	245	160	113
38mm	30/6	39	30	19	14
76mm	24/4	199	149	98	69

EXAMPLE—COMPUTE G'

Compute G' for a 38mm x 0.91mm deck fastened with a 36/9 pattern with supporting joists at 1800mm o/c.

$$G' = \frac{K_2}{K_4 + 0.3(D_{xx} / Span) + 3(K_1)(Span)}$$

$$G' = \frac{191 kN/mm}{3.78 + 0.3(30m/1.8m) + 3(0.982m^{-1})(1.8m)}$$

$$G' = \frac{191 kN/mm}{3.78 + 5 + 5.3} = \frac{191 kN/mm}{14.08} = 13.56 kN/mm$$

Result:
G' = 13.56kN/mm = 13,570kN/m

EXAMPLE—DETERMINE DEFLECTION

Continuing our 40m x 22.5m roof deck example, determine the deflection under calculated seismic loading of 12.1kN/m computed at $R_d R_o = 3.9$.

Solution:
The deflection of the deck is expressed by the following equation:

$$\Delta = \frac{5WL^4}{384EI} + \frac{q_{avg}(L/2)}{G' x 10^6}$$

EI for the deck is taken as E for steel times the moment of inertia computed as $\sum Ad^2$ where A is the area of the diaphragm chord and d is the distance from the centre of gravity of the diaphragm. In most cases with a rectangular diaphragm with equal area diaphragm chords this will reduce to:

$EI = E^*$ (2*Chord Area) * (Diaphragm Width / 2)^2

$EI = 200 \times 10^6 kN/m^2 * 2*(1150 * 10^{-6}m^2)*(11.25m)^2 = 58.22 \times 10^6 kNm^2$

G' is the shear stiffness of the deck diaphragm for shear and provides the shear deflection calculation with 38x0.91 deck supported at 1800mm o/c and connected all flutes and side-lap screwed at 150mm o/c the Hilti™ program gives $K_1 = 0.290$/ft. Converting this to metric we get: $K_1 = 0.95$ /m

G' = 0.0136kN/m (see earlier calculation)

$$q_{mq} = \frac{(Shear\_at\_Support/2)}{Width\_of\_Diaphragm} = \frac{242kN/2}{22.5m} = 5.38kN/m$$

$$\Delta = \frac{5(22.1kN/m)(40m)^4}{384*58.22x10^6\ kN/m^2} + \frac{5.38kN/m*(40m)}{0.0136kN/mx10^6}$$

$$\Delta = 0.013m + 0.016m = 0.029m = 29mm$$

These are the deflections under the SFRS capacity forces for the deck alone. This deflection should be added to the deflection of the brace bays to give the total deflection of the roof relative to the ground. Like all seismic deflections the R_dR_o values used in computing the forces on the structure must be removed according to building code Clause 4.1.8.13(3) by multiplying the calculated deflections by R_dR_o/I_E to give a realistic value for the anticipated deflection.

$$\Delta = (29mm)*\frac{R_dR_o = 3.9}{I_E = 1.0} = 111mm \text{ (Bending = 49 mm while shear = 62mm)}$$

To compare the deflection at the middle of the roof diaphragm to the deflection at the middle of the second floor diaphragm we would have to compute the deflection of the SFRS and the deflection of the floor diaphragm.

As a non-post-disaster building the maximum deflection permitted by the building code would be $0.025*h_s = 0.025*4500mm = 113mm$. Assuming that the deflection of the SFRS and the floor diaphragm are about equal our roof diaphragm is stiff enough but only just. If necessary, the deflection could be reduced by increasing the size of the diaphragm chords or increasing the thickness of the deck material.

CHAPTER 21
A FEW FINAL THOUGHTS

The design of structures is constantly evolving and advancing as we strive to build more dramatic and cost-efficient structures. A designer today not only looks for the weak link in the structure to guard against failure but also for innovation and economy. Advances in technology have allowed us to design more complex buildings faster and more efficiently, but also pose challenges when architects and owners ask for more than stodgy rectangular building designs.

At the same time, codes and standards seem to be constantly getting more restrictive and require increasingly more understanding to implement. The best recent example of such as change would be the seismic requirements of Clause 27. S16.1-84 included no separate seismic clause – seismic was just another lateral load and the concept of capacity design was not considered. Ten years later, S16.1-94 set out a seismic clause of nine-and-a-half pages long covering five structural systems. Thirty years later, S16-14 Clause 27 has grown to 29 pages and contains rules that must be followed for eleven types of structural systems. The complexity of the requirements in the standard reflect the complexity of seismic design, as well as the increased knowledge that comes with full-scale testing and extensive laboratory and theoretical studies following each earthquake in a populated area. A past example of a change that produced fundamental changes in design practice was going to limit states design. In the early 1970s, engineering students in Canada learned steel design using allowable stresses and the *CISC Handbook* included only allowable stress tables. In 1974, the first limit states design edition of S16 was released and by the end of the 1970s limit states design of steel structures had become standard practice in Canada. In fact, the last allowable stress version, S16-69, was retracted officially in 1984. It is only through a commitment to lifelong learning that we can keep up with the evolution of codes and standards, and design practice in general. These changes require continued investment in your knowledge and understanding.

Steel is a wonderful material to work with. It is ductile and homogeneous with a high strength-to-weight ratio, has similar tension and compression strengths, and can be shaped and connected easily to make useful and elegant structures. The nitty-gritty "calculator, pencil, and paper" design of buildings using steel is one of the most pleasing and satisfying professional activities that a structural engineer can do.

It is hoped that after reading and using this book you will have a better insight into the design process and principles of structural steel design, including aspects that facilitate fabrication and erection of the structure. You have seen that building structural design is a holistic experience that considers more than just designing a series of simple span beams. Any building is a series of parts—the horizontal floor systems, the columns, and the lateral system—and these parts must all interact in harmony to form a complete building that meets code requirements and satisfies your client.

It is also hoped that you will continue to learn and advance your knowledge and skills as codes, standards and design practice evolves. The one certainty in the structural design field is that there will be significant changes in design practices in the future, in the same way that there have been in the past. Performance-based seismic design, in which the owner of the building has a greater say in his or her expectations for post-seismic response, is likely to be one of those changes. Advancements in computer-aided design will also change the way we do our work, but there will always be a need for a fundamental understanding of how buildings go together and behave, and for engineers who have the ability to do rapid hand calculations and who can innovatively determine where the design could be improved.

REFERENCES

REFERENCES FOR STEEL DESIGN

- *Canadian Institute of Steel Construction Handbook of Steel Construction*, 11th edition. Toronto: Canadian Institute of Steel Construction, 2016. This book is fundamental for steel design in Canada and is essential for designing steel structures in Canada.
  - The *CISC Handbook* contains the steel standard, referred to as S16-14. This is the steel standard that will soon be recognized in all of Canada, including British Columbia, and this will be the reference code for the book.
  - The *CISC Handbook* also includes commentary on S16.
  - As a new steel handbook comes out every five years there is a temptation to use an outdated version. However these earlier editions of the handbook contain earlier versions of the Code and the tables do not fully reflect the present steels or Code requirements.
  - The latest edition of the *CISC Handbook* is available at most university bookstores. The handbook can also be ordered from the CISC website.
- Part 4 of National Building Code of Canada 2015, issued by the Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, 2015. Alternatively, Part 4 of the 201? British Columbia Building Code (not yet published) or Part 4 of the 201? Vancouver Building Bylaw (not yet published) are essentially identical, even if they do use annoying typefaces and page layout. Most structural design is done with reference to only Part 4 of the Building Code.
- Supplement to the National Building Code of Canada 2015. The supplement is a good reference to have to understand Code loadings and should be required reading for every practising structural engineer. The Supplement was published for NBCC 2015 in 2017.
- G.L. Kulak and G.Y. Grondin. *Limit States Design in Structural Steel*, 9th edition. Canadian Institute of Steel Construction, 2011. (This is a good reference on many of the code vertical design issues with worked examples. The 9th edition uses S16-09 and NBCC 2010 as its reference codes.) Updates and corrections to this reference can be found on the CISC website.
- Manufacturers' literature such as from VicWest® on deck properties and Open Web Steel Joist manufacturer's design tables. This material is available through the internet by visiting the manufacturer's website.
- Web resources: The CISC website at cisc-icca.ca gives some technical information and can be used to order their own publications as well as some of the publications referenced in the Code. CISC also makes some software available through their website.
- More information about steel sustainability can be found in the following references:
  - The Steel Recycling Institute website at recycle-steel.org.
  - CISC has an excellent section on their website on sustainability including links to other sites.
  - The World Steel Association dedicates a portion of their website to green aspects of steel and its production.

  - The August 2000 edition of *Atlantic Monthly* magazine had a cover article on the downside of recycling steel ships under Third World conditions which brought attention to the safety and environmental issues associated with breaking up ships in developing nations. The article is no longer available online on the *Atlantic Monthly* website but a quick search on any search engine for "ship breaking" will bring up several sources try Wikipedia for a brief background and list of sources.

REFERENCES FOR SECTION CLASS AND UNBRACED BEAMS

- *Canadian Institute of Steel Construction Handbook of Steel Construction*, 11th edition. Toronto: Canadian Institute of Steel Construction, 2015.
  - Section class requirements for members given in Clause 11.1, the commentary to S16 that is in the handbook has an excellent table and illustrated figures.
  - Section class for members in bending is given in Table 5-1.
- Part 4 of National Building Code of Canada 2015 (NBCC) issued by the Associate Committee on the National Building Code, National Research Council of Canada, Ottawa 2015. The Building Code gives snow loading criteria. (As an alternative, Part 4 of the Vancouver Building Bylaw or British Columbia Building Code would be equally appropriate.)
- User's Guide – NBCC 2015 Structural Commentaries (Part 4 of Division B) Canadian Commission on Building and Fire Codes (NRC) 2015. The commentary also has an excellent background to the seismic provisions in the Code along with additional information and explanation on seismic design and load factors that will be of use to structural designers.
- Designing Roofs for Rain Loads. October 1995, Part 4 Committee of the Association of Professional Engineers and Geologists of British Columbia. Hard to find this reference now; however, much of its material has been included in this book.

REFERENCES FOR FRAMING OF STEEL STRUCTURES

- *Canadian Institute of Steel Construction Handbook of Steel Construction*, 11th edition. Toronto: Canadian Institute of Steel Construction, 2015. This will be necessary for the beam tables and code references.
- Part 4 of National Building Code of Canada 2015 (NBCC) issued by the Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, 2015. The Building Code gives snow loading criteria. (Alternatively, Part 4 of the Vancouver Building Bylaw or British Columbia Building Code would be equally appropriate.) Design loadings for floors can be found in Part 4.
- User's Guide – NBCC 2015 Structural Commentaries (Part 4 of Division B). National Research Council of Canada, Ottawa, 2016. The Structural Commentaries used to be the only source for snow and wind load information but this is now included directly in the NBCC 2015.

REFERENCES FOR DECK DESIGN

- *Canadian Institute of Steel Construction Handbook of Steel Construction*, 11th edition. Toronto: Canadian Institute of Steel Construction, 2015, contains some background information on steel deck, including a rough guide to profiles in the CISC Code of standard practice. This chapter also contains a list of available zinc coatings and a list of Canadian Sheet Steel Building Institute (CSSBI) publications.
- *Design and Construction of Composite Floor Systems* by E. Chien and J. Keith Ritchie (Toronto: Canadian Institute of Steel Construction, 1984). This is a good reference for floor construction

and Chapter 1 covers several items concerning composite deck. Many of the topics covered in the subsequent chapters of this reference will be more appropriate to a discussion of composite beams.

- Most designers refer to the manufacturer's literature when designing steel deck. Manufacturers that produce design information for their deck products include VicWest®, Mercury Metals, Canadian Metal Rolling Mills, and Canam. In addition the deck manufacturers will provide background information if requested; for example, VicWest® has a composite deck design manual that contains formulas, details and examples. Deck capacity charts from VicWest® are available on their website.
- The Canadian Sheet Steel Building Institute (CSSBI) produces several useful documents on decking. These are available through their website (cssbi.ca), several for free download. In particular, the following documents are of use:
  - CSSBI Standard for sheet steel roof deck (CSSBI 10M-06), September 2006
  - CSSBI Standard for composite steel deck (CSSBI 12M-06), September 2006
  - CSSBI Criteria for the design of composite slabs (CSSBI S3-03), September 2003
- VicWest® has a website that includes deck charts for download. The site can be found at Vicwest.com; portions of VicWest® tables are reproduced in this book, and the use of these charts for educational purposes is acknowledged and appreciated.
- ComSlab® is now a product of Bailey Metal Products and information can be found on the Bailey Metal Products website under "Products."

REFERENCES FOR LIMIT STATES DESIGN

- *National Building Code of Canada 2015*. Ottawa: National Research Council of Canada, 2015.
- Commentary A, Limit States Design. In *User's Guide – NBCC 2015 Structural Commentaries*, 3rd edition. Ottawa: National Research Council of Canada, 2016.
- *Canadian Institute of Steel Construction Handbook of Steel Construction*, 11th edition. Toronto: Canadian Institute of Steel Construction, 2015.
  - Clause 9 of S16 covers stability of structures and members (this has been brought forward from its original position of Clause 20 in S16.1-94 to emphasize its importance).
- G.L. Kulak and G.Y. Grondin. *Limit States Design in Structural Steel*, 9th edition. Toronto: Canadian Institute of Steel Construction, 2011. In particular, for limit state design:
  - There is an example of the design of a cantilever girder system roof in accordance with NBCC 2010. The effect of companion action wind is considered with the snow load, for consideration of the stability of the cantilever girder the reader is referred to work by Kennedy and Essa (referenced below).
  - There is a discussion and example of frame stability under wind loading, including the effect of wind load and notional loads.
- *Guide to Stability Design Criteria for Metal Structures*, 5th edition. Edited by Theodore Galambos; John Wiley & Sons, 1998. In particular, for information on cantilever beam stability, see:
  - There is a discussion of cantilever beam systems and the effect of the tip and root restraint on the overall lateral stability of the beam.
  - The book shows how to calculate the stability of the interior beam.
- *Guide to Stability Design Criteria for Metal Structures*, 6th edition. Edited by Ronald D. Ziemian, John Wiley & Sons, 2010. This edition is referenced in S16 but is a lot more complex to wade through than the 5th edition and leaves out some of the aspects of cantilever beams that were covered in the 5th edition. For example, using the 6th edition, it is not possible to compute the critical moment for a cantilever that did not have both top and flanges supported at the root.
- *Roof Framing with Cantilever (Gerber) Girders and Open Web Steel Joists*. Toronto: Canadian Institute of Steel Construction, July 1989. This was published in response to the Metrotown Save-On Foods roof failure but unfortunately the booklet is no longer in publication though you might find a copy tucked away in many engineering offices. Two very important pages from the publication have been reproduced as part of this text.
- D.J. Laurie Kennedy and Hesham S. Essa. *Distortional Buckling of Steel Beams*, April 1993, University of Alberta Department of Civil Engineering Structural Engineering Report No. 185.
  - Covers the theory of the cantilevered framing system and also the 33 full-scale tests that were done as part of the study. In particular, Chapter 7 "Design procedures for laterally unsupported beams" will be of use in design.
- S16 Annex K on Fire Rating.
- R.G. Gewain, N.R. Iwankiw, F. Alfawakhiri, G.S. Frater. *Fire Facts for Steel Buildings*, 1st edition. Toronto: Canadian Institute of Steel Construction, April 2006. This publication is a free PDF download from the CISC website.

REFERENCES FOR AXIALLY LOADED MEMBERS

- *Canadian Institute of Steel Construction Handbook of Steel Construction*, 11th edition. Toronto: Canadian Institute of Steel Construction, 2015. This book contains several references and aids for the design of columns:
  - Clause 13 and the commentary to Clause 13 cover the design of axial members, including beam-columns.
  - Clause 25 covers base plates.
  - The green pages of the *CISC Handbook* offer column capacity tables for design for wide-flange columns and HSS columns.
  - Column base plates are covered in their own section of the *CISC Handbook*.
  - There is a good example of beam-column calculation.
  - S16 is used as a reference to axial capacity of single angle braces.
- G.L. Kulak and G.Y. Grondin. *Limit States Design in Structural Steel*, 9th edition. Toronto: Canadian Institute of Steel Construction, 2011. (Chapter 8 is a good reference on beam-columns, including worked examples according to S16-09. Chapter 4 covers members with axial force without bending. Section 3.2 covers tension members.)

REFERENCES FOR OPEN WEB STEEL JOISTS

- *Canadian Institute of Steel Construction Handbook of Steel Construction*, 11th edition. Toronto: Canadian Institute of Steel Construction, 2015.
- Clause 16 Open-Web Steel Joists, including commentary. In *User's Guide – NBCC 2015 Structural Commentaries*, 3rd edition. Ottawa: National Research Council of Canada, 2016.
- Manufacturers' literature produced by the manufacturers of Open Web Steel Joists, including span tables and other material. This material is available on the manufacturers websites but it may be necessary to register to obtain access to technical information.

REFERENCES FOR HOLLOW STRUCTURAL STEEL TRUSSES

- *Canadian Institute of Steel Construction Handbook of Steel Construction*, 11th edition. Toronto: Canadian Institute of Steel Construction, 2015, including the following:
  - Hollow Structural Section Connections
  - Clause 15 for Trusses, including commentary.
- Packer, J.A., and J.E. Henderson. *Hollow Structural Section Connections and Trusses: A Design Guide*, 2nd edition. Willowdale, Ontario: Canadian Institute of Steel Construction, 1997. This is the essential resource for designing HSS trusses and every engineer who is doing production design of HSS trusses should have a copy of this book available.

REFERENCES FOR CONNECTION DESIGN

- The main reference for connection design in Canada is the *Canadian Institute of Steel Construction Handbook of Steel Construction*, 10th edition. Toronto: Canadian Institute of Steel Construction, 2015, including the following:
  - General information on connections, including worked examples, is given in Part 3. The yellow pages in the middle of the section make this information easier to find.
  - Code requirements for bolted and welded connections are given in Clause 13.11 to 13.14.
  - Clause 21 covers connections.
  - Clause 22 covers bolting details.
  - Clause 23 covers structural joints using a variety bolts.
  - Clause 24 covers welding.
  - Additional information and requirements on welding can be found in CSA Specification W59-03 (R2008) Welded Steel Construction (Metal Arc Welding).
  - Beam load tables (blue pages) in the *CISC Handbook* are used to determine the connection design forces.
  - Many of the responsibilities for structural steel erection are laid out in the CISC Code of Standard Practice for Structural Steel which is contained in the *CISC Handbook*.
  - Information on welding, including weld symbols, is given in the handbook.
  - Information on bolts, including dimensions and clearances, is given in the handbook.
- G.L. Kulak and G.Y. Grondin. *Limit States Design in Structural Steel*, 9th edition. Toronto: Canadian Institute of Steel Construction, 2011, including the following:
  - Chapter 9 covers connections.
- The American Institute of Steel Construction (AISC) has information on connection design in their handbooks and website but it should be noted that these should be regarded only as a secondary source of information. All structural steel for use in Canada must be detailed to the Canadian code.

REFERENCES FOR SEISMIC DESIGN

- National Building Code of Canada 2015, issued by the Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, 2015. Clause 4.1.8 (Live Loads due to Earthquakes). (This is not just a suggested reference. This is a must-have document if doing seismic design in Canada.)
- Supplement to the National Building Code of Canada 2015. (Structural Commentaries – Commentary J covers seismic design while commentary L covers the application of NBCC Part 4 to the structural evaluation and upgrading of existing structures.) This document is only slightly less important than the Code itself.
- *Canadian Institute of Steel Construction Handbook of Steel Construction*, 11th edition, Toronto: Canadian Institute of Steel Construction, 2015. The *CISC Handbook* contains no specific worked examples on seismic design but it does contain S16 and the commentary on S16, both of which are fundamental to seismic design. S16 Clause 27 covers seismic design. Several changes are made to Clause 27 with each edition of S16 and it is important to have the current version.
- Canadian Sheet Steel Building Institute (CSSBI), *Design of Steel Deck Diaphragms*, 3rd edition, January 2006. This manual can be purchased for $50 from the CSSBI website at cssbi.com. The tables use $\emptyset = 0.5$, which is acceptable for decks fastened with puddle welds or button punching but is overly conservative for decks fastened with mechanical fasteners and side-lap screws as there decks now use $\emptyset = 0.6$.
- University of Alberta Department of Civil Engineering Structural Engineering Report No. 219, Seismic Evaluation of Steel Buildings with Concentrically Braced Frames by Manoj S. Medhekar and D.J. Laurie Kennedy.
- A basic background on earthquakes and their effects can be found in many excellent websites, including those from the following sites:
  - Earthquakes Canada website at earthquakescanada.nrcan.gc.ca/
  - USGS website at earthquake.usgs.gov/
  - FEMA (Federal Emergency Management Association) at fema.gov/
- A good primer on earthquake behaviour and effects can be found in *Earthquakes* by Bruce A. Bolt. New York: W.H. Freeman & Company, 1993.
- *At Risk Earthquakes and Tsunamis on the West Coast* by John Clague, Chris Yorath, Richard Franklin and Bob Turner. Vancouver: Tricouni Press, 2006. This 200-page book is a good background to the seismic conditions in coastal British Columbia written by local geologists. The book is well illustrated.
- Hilti™ PROFIS DF Deck diaphragm program is downloadable from their website provided you have registered. This program is very useful for determining the shear capacity of metal roof deck diaphragms.
- Filiatrault, Andre, Robert Tremblay, Constantine Christopoulos, Bryan Folz, and Didier Pettinga. *Elements of Earthquake Engineering and Structural Dynamics*, 3rd edition. Montreal: Presses Internationales Polytechnique, 2013. This is an excellent background to earthquake analysis and design with lots of helpful examples and illustrations on how to do both the analysis of the forces and the design of the elements in steel, concrete and wood.

REFERENCES FOR DECK DIAPHRAGM TESTING

- Tremblay, R., C.A. Rogers, E. Martin, W. Yang, W. (2004). "Analysis, testing, and design of steel roof deck diaphragms for ductile earthquake resistance." *Journal of Earthquake Engineering*, 8(5), 775–816.
- Essa, H.S., R. Tremblay, C.A. Rogers. (2003). "Behaviour of roof deck diaphragms under quasistatic cyclic loading." *Journal of Structural Engineering*, ASCE, 129(12), 1658–66.
- Rogers, C.A., R. Tremblay. (2003). "Inelastic seismic response of side-lap fasteners for steel roof deck diaphragms." *Journal of Structural Engineering*, ASCE, 129(12), 1637–46.
- Rogers, C.A., R. Tremblay. (2003). "Inelastic seismic response of frame fasteners for steel roof deck diaphragms." *Journal of Structural Engineering*, ASCE, 129(12), 1647–57.
- Pranquet, J., R. Massarelli, K. Shrestha, R. Tremblay, and C.A. Rogers. (2010). "Dynamic Tests of 0.76mm and 0.91mm Steel deck diaphragms for single-storey buildings." Proceedings of 9th U.S. National and 10th Canadian Conference on Earthquake Engineering, July 25–29, 2010 Toronto, Canada, Paper 1008.
- Rogers, Colin A., and Robert Tremblay. (2010). "Impact of Diaphragm Behavior on the seismic design of Low-Rise Steel Buildings." *Engineering Journal* AISC, 1st Quarter 2010.

REFERENCES FOR SEISMIC SYSTEMS

- *Canadian Institute of Steel Construction Handbook of Steel Construction*, 11th edition, Toronto: Canadian Institute of Steel Construction, 2015.
  - There are no specific numerical examples in the *CISC Handbook* that relate directly to seismic design; however, all the tables we have been using in static design still apply and design in steel for seismic loads in Canada without the current *CISC Handbook* would be beyond comprehension.
  - S16 Clause 27 covers seismic design. The seismic section of S16 is the most rapidly changing section of the standard and care should be taken to use the most up-to-date version of the standard.
  - See also the commentary in the *CISC Handbook*, which is a fundamental part of understanding the requirements and clauses in the Code and the commentary, should be read in conjunction with the Code.

- National Building Code of Canada 2015, Clause 4.1.8 (Live Loads due to Earthquakes) issued by the Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, 2015.
- User's Guide – NBCC 2015 Structural Commentaries (Part 4 of Division B), Commentary J, refers to seismic loading. In particular, see the sections in the commentary covering SFRS force reduction factors and the equivalent static loading that we use for evaluating low-rise steel buildings.
- Tremblay, R., M. Bruneau, R. Driver, A. Metten, C.J. Montgomery, and C.A. Rogers. *Seismic Design of Steel Structures in Accordance with CSA S16-09*. Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering. July 25–29, 2010, Toronto, Canada.
- M. Bruneau, C.-M. Uang, and A. Whittaker. *Ductile Design of Steel Structures*, McGraw Hill, 1998, 500 Pages. Second edition 2011. This is a great reference on the design of steel with discussions on philosophy of steel structures for seismic design, several worked examples and photographs illustrating what can go wrong if the systems are not detailed well.

REFERENCES FOR BRACE BAYS

- El-Tayem, A.A., and S.C. Goel. "Effective Length Factor for the Design of X-Bracing Systems." *AISC Engineering Journal* (1986), 1st Quarter: 41–45.
- Korol, R.M. "Shear Lag in Slotted HSS Tension Members." *Canadian Journal of Civil Engineering* (1996), 23: 1350–54.
- Picard, A., and D. Beaulieu. "Design of Diagonal Cross Bracings. Part 1: Theoretical Study." *AISC Engineering Journal* (1987), 3rd Quarter: 122–26.

REFERENCES FOR MOMENT FRAMES

- *Canadian Institute of Steel Construction Handbook of Steel Construction*, 11th edition, Toronto: Canadian Institute of Steel Construction, 2015.
  - Some information on moment connections is given in Chapter 3 on connections.
  - S16 Clause 27 covers seismic design.
  - S16 Annex J Ductile Moment Resisting Connections. In earlier editions of the code this contained considerable material but this has been moved into the body of S16 and refereaces the CISC guidebook on moment connections.
- Canadian Institute of Steel Construction. *Moment Connections for Seismic Applications*. 2015. This is the second edition of this guidebook and it takes advantage of several innovations and converts to Canadian codes much of the work on moment connections done by AISC. This publication can be downloaded for $40 from the CISC website.
- National Building Code of Canada 2015, Clause 4.1.8 (Live Loads due to Earthquakes), issued by the Associate Committee on the National Building Code, National Research Council of Canada, Ottawa, 2015.
- User's Guide – NBCC 2015 Structural Commentaries (Part 4 of Division B), Commentary J, covers seismic design.
- U.S. Federal Emergency Management Agency. FEMA-350 Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings. June 2000. This publication is available for free download from the FEMA website. FEMA-355 State of the Art report on Connection design was published in 2000. The report can also be found by going through the FEMA website and finding Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings. Other FEMA reports are available for download over the Internet. FEMA has done a truly amazing job at researching seismic issues and distributing the information to engineers. Their free distribution of their research publications to Canadian engineers is very generous.

- James Ricles, Xiaofang Zhang, Lo-Wu Lu, and John Fisher. *Development of Seismic Guidelines for Deep-Column Steel Moment Connections*. ATLSS Report No. 04-13. June 2004. Advanced Technology for Large Structural Systems (ATLSS) Engineering Research Center, Lehigh University, Bethlehem, PA. Available in PDF format from AISC website.
- American Institute of Steel Construction (AISC). *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*. ANSI/AISC 358. December 13, 2005. Available on the AISC website.

REFERENCES FOR COMPOSITE BEAMS

- *Canadian Institute of Steel Construction Handbook of Steel Construction*, 11th edition. Toronto: Canadian Institute of Steel Construction, 2015.
  - S16 Clause 17 covers composite beams along with the commentary.
  - Pages 5-20 to 5-67 contains a discussion of composite beams, tables on shear capacities of the studs, worked examples and tables to make some aspects of composite beam design more direct.
  - Annex H covers shrinkage of concrete in composite beams.
- E. Chien and J. Keith Ritchie. *Design and Construction of Composite Floor Systems*. Toronto: Canadian Institute of Steel Construction, 1984.
  - This is a good reference covering many aspects of composite floor design but is getting hard to find.
- G.L. Kulak and G.Y. Grondin. *Limit States Design in Structural Steel*, 9th edition. Toronto: Canadian Institute of Steel Construction, 2011.
  - Chapter 6 covers composite beams, including worked examples.

REFERENCES FOR VIBRATION ANALYSIS

- *Canadian Institute of Steel Construction Handbook of Steel Construction*, 11th edition. Toronto: Canadian Institute of Steel Construction, 2015.
  - S16 Annex E covers floor vibration. This appendix used to be several pages long with formulas and charts but now mostly provides good references.
- Thomas Murray, David Allen, and Eric Ungar. *Floor Vibrations Due to Human Activity*, AISC-CISC Steel Design Guide Series Number 11, 1997 (Revised October 2003).
  - This is good reference, including worked examples. The spreadsheet examples given in the text were developed based on the examples in this reference.
- NBCC 2015 Structural Commentaries (Part 4)
  - Commentary A covers serviceability issues including floor vibrations.
- Two good web references profile London's Millennium Bridge and its swaying, including the research and solution. The project website of Arup Engineers discusses the challenges involved in the design, including oscillation, at arup.com/MillenniumBridge. You can watch the bridge wobble and read an in-depth report on the project on the BBC News website by clicking on "Special Reports" in the navigation buttons at the top of the home page; then scrolling down to find the "Archive by year." The Millennium Bridge special report can be found in the 2001 archive under the heading "UK."

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ABOUT THE AUTHORS



Andy Metten is a practising structural engineer and partner in the Vancouver-based structural engineering firm of Bush, Bohlman & Partners. Over the past 30 years, he has been the design engineer on several steel buildings, including the Vancouver International Airport and the U.S. Terminal in Nassau, Bahamas, and has designed the steel roofs of many concrete buildings. Andy is still closely involved in day-to-day design of structures from conceptual design through field services. Andy Metten has practised engineering since graduation from the University of British Columbia with a bachelor's and master's degrees in structural engineering in 1978 and 1981. He is currently a member of the Standing Committee for Earthquake Design for the National Building Code of Canada and the S16 structural steel design committee for Canada. For the past 12 years, he has also taught an evening structural steel design course that has been well received by more than 500 participants. He is a Fellow of the Canadian Society for Civil Engineering.



Robert Driver is a professor and the associate chair of the Department of Civil and Environmental Engineering at the University of Alberta. He has a total of 30 years of experience in the steel fabrication industry, structural engineering consulting, applied engineering research, and education, having taught structural engineering undergraduate and post-graduate courses in Canada, the United States, and South Africa. Robert chairs technical subcommittees for CSA Standards S16, *Design of Steel Structures*, and S850, *Design and Assessment of Buildings Subjected to Blast Loads*, and is a member of the Education and Research Council of the Canadian Institute of Steel Construction. He has published approximately 200 papers and technical reports, supervised over 50 graduate students and post-doctoral fellows, and has been the recipient of numerous national and international awards for both research and teaching. He is a Fellow of the Canadian Society for Civil Engineering.